Chapter 9
Structural Deformation Monitoring Surveys

9-1. General

The Corps of Engineers has constructed hundreds of dams, locks, levees, and other flood control structures which require periodic surveys to monitor long-term movements and settlements or short-term deflections and deformations. In USACE, these types of surveys are generally referred to as "PICES Surveys" -- an acronym which derives from the directive Engineer Regulation ER 1110-2-100, Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures. This chapter provides background guidance on the general requirements for these surveys and describes some of the instrumentation and techniques available with which to perform deformation surveys. Subsequent chapters provide specific accuracy standards, specifications, frequencies, and procedural methods for performing various types of surveys.

a. *Structural deformation.* Dams, locks, levees, embankments, and other flood control structures are subject to external loads that cause deformation and permeation of the structure itself, as well as its foundations. Any indication of abnormal behavior may threaten the safety of the structure. Careful monitoring of the loads on a structure and its response to them can aid in determining abnormal behavior of that structure. In general, monitoring consists of both measurements and visual inspections, as outlined in ER 1110-2-100. To help to ensure the safe monitoring of a dam, it should be permanently equipped with proper instrumentation according to the goals of the observation, structure type and size, and site conditions. Guidance on the instrumentation required to measure internal loads on a structure can be found in USACE technical manuals dealing with concrete structures, earth and rock fill dams, and similar structures.

b. *Concrete structures.* It should be intuitive that deformations and periodic PICES observations will vary according to the type of structure. Differences in construction materials is one of the larger influences on how a structure deforms. For example, concrete dams deform differently than earthen or embankment dams. For concrete dams and other concrete flood control devices, deformation is mainly elastic and highly dependent on reservoir water pressure and temperature variations. Permanent deformation of the structure can sometimes occur as the subsoil adapts to new loads, concrete aging, or foundation rock fatigue. Such deformation is not considered unsafe if it does not go beyond a predetermined critical value. Therefore, PICES observations are typically configured to observing relatively long-term movement trends, including abnormal settlements, heaving, or lateral movements. Conventional geodetic survey methods from external points and of centimeter-level accuracy are sufficient to monitor these long-term trends. Highly accurate, short-term deflections or relative movements between monoliths due to varying temperature or hydraulic loading are more rarely required. These may include crack measurements or relative movements between monoliths over different hydraulic loadings. Relative movement deflections to the 0.01-inch accuracy level are common.

c. *Earthen embankment structures.* Earthen or embankment dams and levees obviously will deform altogether differently than concrete ones. With earthen dams, the deformation is largely characterized as more permanent. The self weight of the embankment and the hydrostatic pressure of the reservoir water largely force the fill material (and, in turn, the foundation if it too consists of soil) to settle, resulting in a vertical deflection of the structure. The reservoir water pressure also causes permanent horizontal deformation perpendicular to the embankment centerline. With earthen dams, elastic behavior is slight. PICES survey accuracy requirements are less rigid for earthen embankments, and traditional construction survey methods will usually provide sufficient accuracy. Typical PICES surveys include periodic measurement of embankment crest elevations and slopes to monitor settlements and slope stability. For embankment structures, survey accuracies of 0.1 foot are usually sufficient for monitoring long-term settlements and movements.

d. *Long-term deformation monitoring.* Depending on the type and condition of structure, PICES monitoring systems may need to be capable of measuring both long-term movement trends or short-term loading deformations. Long-term measurements are far more common and somewhat more complex given their external nature. Long-term monitoring of a structure's movement typically requires observations to monitoring points on the structure from external reference points. These external reference points are established on stable ground well removed from the structure or its construction influence. These external reference points are inter-connected and are termed the "Reference Network." The reference network must also be monitored at less-frequent intervals to ensure these

reference points have not themselves moved. Traditional geodetic survey instruments and techniques may be employed to establish and monitor the reference network points, as described in Chapter 10. Procedures for observing periodic PICES observations from the reference points to the structure targets are covered in Chapter 12.

9-2. Deformation Monitoring Survey Techniques

a. The general procedures to monitor the deformation of a structure and its foundation involve measuring the spatial displacement of selected object points (i.e., target points) from reference points, themselves controlled in position. When the reference points are located in the structure themselves, only relative deformation can be determined. Micrometer joint measurements are relative observations. Absolute deformation or displacement can be determined if the reference points are located outside the actual structure, in the foundation or surrounding terrain, and beyond the area that may be affected by the dam or reservoir. Subsequent periodic observations are then made relative to these absolute reference points. Assessment of permanent deformations requires absolute data.

b. In general, for concrete dams it is ideal to place the reference points in a rock foundation at a depth unaffected by the reservoir. Once permanently monumented, these reference points can be easily accessed to perform deformation surveys with simple measurement devices. Fixed reference points located within the vicinity of the dam but outside the range of its impact are essential to determination of the deformation behavior of the structure. Thus, monitoring networks in the dam plane should be supplemented by and connected to triangulation networks and vertical control whenever possible.

c. The monitoring of dam or foundation deformation must be done in a manner such that the displacement is measured both horizontally and vertically (i.e., measurement along horizontal and vertical lines). Such measurements must include the foundation and extend as far as possible into it. Redundancy is essential in this form of deformation monitoring and is achieved through measuring at the points intersecting the orthogonal lines of the deformation network.

d. If a dam includes inspection galleries and shafts, deformation values along vertical lines can be obtained by using both hanging and/or inverted plumb lines and along horizontal lines by traverses; both are standard practice for deformation monitoring. Where there are no galleries or shafts (e.g., embankment dams, thin arch dams, or small gravity dams), the same result can be achieved by an orthogonal network of survey targets on the downstream face. These targets are sighted by angle measurements, typically combined with optical distance measurements, from reference points outside the dam.

e. A more routine, less costly, and more frequent monitoring process can be employed to monitor the short-term behavior of dams by simply confining observation to trends at selected points along the crest and sometimes vertical lines. Such procedures involve angle measurement or alignment (supplementing the measuring installation) along the crest to determine horizontal displacement, and leveling to determine vertical displacement. Even with this monitoring process, it is essential to extend leveling to some distance beyond the abutments. Alternative methods to that described include settlement gauges, hose leveling devices, or extensometers.

9-3. Accuracy Requirements for PICES Surveys

Table 9-1 provides general guidance on the accuracy requirements for performing PICES surveys. These represent either absolute or relative movement accuracies on structure target points that should be attained from survey observations made from external reference points. The accuracy by which the external reference network is established and periodically monitored for stability should exceed these accuracies. Many modern survey systems (e.g., DGPS, electronic total stations, bar-code levels, etc.) are easily capable of meeting or exceeding the accuracies shown below. However, PICES accuracy criteria must be defined relative to the particular structure's requirements, not the capabilities of a survey instrument or system.

a. For example, a good electronic total station system can measure movement in an embankment to the 0.005-foot level. Thus, a long-term creep of, say 3.085 feet, can be accurately measured. However, the only significant aspect of the 3.085-foot measurement is the fact that the embankment has sloughed "3.1 feet" -- the 0.001-foot resolution (precision) is not significant and should not be observed even if available with the equipment.

b. Relative crack or monolith joint micrometer measurements can be observed and recorded to 0.001-inch precisions. This precision is not necessarily representative of an absolute accuracy, given the overall error budget in the micrometer measurement system, measurement plugs, etc. Hydraulic load and temperature influences can radically change these short-term micrometer measurements at

Table 9-1
Typical Accuracy Requirements for PICES Surveys

Concrete Structures
Dams, Outlet Works, Locks, Intake Structures:

Long-term movement (Geodetic survey methods)	10 mm
Relative short-term deflections Crack/joint movements Monolith alignment (Precision micrometer alignments)	0.01 in. (0.2 mm)
Vertical stability/settlement (Precise geodetic leveling)	2 mm

Embankment Structures
Earth-Rockfill Dams, Levees:

Slope/crest stability (Total station/DGPS)	0.1 foot
Crest alignment (Total station/DGPS)	0.1 foot
Settlement measurements (Differential leveling)	0.05 foot

Control Structures
Spillways, Stilling Basins, Approach/Outlet Channels, Reservoirs

Scour/erosion/silting (Hydrographic surveys)	0.2 to 0.5 foot

the 0.01- to 0.02-inch level, or more. Attempts to observe and record micrometer measurements to a 0.001-inch precision with a ±0.01-inch temperature fluctuation is wasted effort.

9-4. PICES Classification for Determining Monitoring Frequency

The following presents general guidance in determining the frequency which periodic monitoring surveys must be performed, giving variable structure type, risk, and age.

Class I: HIGH RISK STRUCTURES IN DISTRESS. The high risk of Class I structures warrants continuous monitoring of the structure.

Type A: POTENTIAL FAILURE IMMINENT. Gather data as can/if can/if prudent. Data are very valuable for later analysis of why the structure failed. Use any method available to gather information and data without risk of life or interference in processes ongoing to save the structure and/or alert the population at risk.

Type B: POTENTIAL FAILURE SUSPECTED. Monitor structure continuously. After potential solution to save structure is applied, continue with continuous monitoring until it is determined that structure is stabilized.

Type C: DAMS OR RESERVOIRS UNDERGOING INITIAL IMPOUNDMENT. Gather ground truth data before impoundment procedures start. Monitor continuously until failure is suspected or until impoundment procedures have halted. Continue monitoring continuously until it is determined that structure has stabilized and will maintain as planned under load.

Class II: MEDIUM RISK STRUCTURES NOT IN DISTRESS. Such structures are of a category of risk such that monitoring every other year is prudent. Structures of this category are stable, but whose failure would:

- affect a population area;

- result in a high dollar loss of downstream property;

- cause a devastating interruption of the services provided by the structure.

Type A: LARGE STRUCTURES.
i. Reservoirs.
ii. Dams with power plants.

Type B: SMALLER STRUCTURES.
i. Reservoirs and dams.
ii. Dams without power plants.
iii. Locks.

Class III: LOWER RISK STRUCTURES NOT IN DISTRESS. Such structures are of a category of risk such that monitoring every other year is prudent. Structures of this category are stable, but whose failure would:

- not affect a population area;

- not result in a high dollar loss of downstream property;

- not cause a devastating interruption of the services provided by the structure.

Type A: LARGE STRUCTURES.
  i. Reservoirs.
  ii. Dams with power plants.

Type B: SMALLER STRUCTURES.
  i. Reservoirs and dams.
  ii. Dams without power plants.
  iii. Locks.

Table 9-2 summarizes the PICES structure classification for determining monitoring frequency.

9-5. Dam Safety

a. Minimizing the risk of dam failure is a primary function of USACE employees involved with PICES work. Existing USACE dams and reservoirs must be periodically inspected so that their structural soundness can be evaluated and reevaluated.

b. In the course of time, dam structures may take on anisotropic characteristics. Internal pressures and paths of seepage may develop. Usually the changes are slow and not readily discerned by visual examination. Continuous monitoring of a dam's performance will usually ensure detection of any flaws which may lead to failure. This must be done by personnel who know the signs of distress. The best knowledge of the forces which cause failure are gained by studying the postmortems of the failed structures themselves.

c. Analysis of the performances for the various types of dams will show their relative suitability for conditions which may be encountered at a given site. Each type can be related generally to a certain mode of failure. A gravity dam may collapse only in the section which is overstressed. A buttress dam may fall in domino fashion through the successive collapse of its buttresses. The rupture of an arch may be sudden and complete. Failure

of an embankment may be relatively slow, with erosion progressing laterally and downward and then accelerating as the floodwaters rip through the breach.

d. The records of dams indicate that earthfills have been involved in the largest number of failures, followed by gravity dams, rockfills, and multiple and single arches. Considering the percentages, however, the arch dams show a higher failure rate. Studies show that there are two predominant causes of damage which are:

- Construction defects.

- Interstitial water that is inadequately controlled.

9-6. Foundation Problems in Dams

a. Foundation problems may lead to breaching of the dam. Differential settlement, sliding, high piezometric pressures, and uncontrolled seepage are common evidences of foundation distress. Cracks in the dam, even minor ones, can indicate a foundation problem. Potential erosion of the foundation must be considered. Clay or silt in weathered joints can preclude grouting and eventually swell the crack enlarging it and causing further stress. Foundation seepage can cause internal erosion or solution. Erosion can leave collapsible voids. Actual deterioration may be evidenced by increased seepage, by sediment in seepage water, or an increase in soluble materials disclosed by chemical analyses. Materials vulnerable to such erosion include such materials as dispersive clays, water reactive shales, gypsum, and limestone.

b. Pumping from underground can cause foundation settlement as the supporting water pressure is removed or the gradient changed. Loading and wetting will also cause the pressure gradient to change, and may also cause settlement or shifting. The consequent cracking of the

Table 9-2
PICES Structure Classification

Structures in Distress		Structures Not In Distress			
Class I: High Risk		Class II: Medium Risk		Class III: Lower Risk	
Continuous Monitoring		Monitor Yearly or Every Other Year		Monitor Every Other Year	
Type A	Potential Failure Imminent	Type A	Large structures	Type A	Large structures
Type B	Potential Failure Suspected				
Type C	Dams or Reservoirs Undergoing Initial Impoundment	Type B	Smaller structures	Type B	Smaller structures

dam can create a dangerous condition, especially in earthfills of low cohesive strength.

c. Foundations with low shear strength or with seams of weak materials such as clay or bentonite may be vulnerable to sliding. Shear zones can also cause problems at dam sites. The bedding plane zones in sedimentary rocks and foliation zones in metamorphics are two common problems.

9-7. Seepage

a. Water movement through a dam or through its foundation is one of the important indicators of the condition of the structure and may be a serious source of trouble. Seeping water can chemically attack the components of the dam foundation. Constant attention must be focused on any changes such as in the rate of seepage, settlement, or in the character of the escaping water. Adequate measurements must be taken of the piezometric surface within the foundation and the embankment, as well as any horizontal or vertical distortion of the abutments and the fill. Generally, differential settlements caused by the dissolving of solid material develop slowly enough to provide advance warning of the need for any remedies.

b. Any leakage at an earth embankment may be potentially dangerous, since rapid erosion may quickly enlarge an initially minor defect.

9-8. Erosion

Embankments may be susceptible to erosion unless protected from wave action on the upstream face and surface runoff on the downstream face. Riprap armor stone on the upstream slope of an earthfill structure can protect against wave erosion, but can become dislodged due to wave action. This deficiency can usually be detected and corrected before serious damage occurs.

9-9. Embankment Movement

a. In an older embankment dam, the condition of materials may vary considerably. There may be small or extensive areas of low strength. Location of these weaknesses must be a key objective of the evaluation of such dams. Soluble materials are sometimes used in construction, and instability in the embankment will develop as these materials are dissolved over time.

b. Adverse conditions which deserve attention include: poorly sealed foundations, cracking in the core

zone, cracking at zonal interfaces, soluble foundation rock, deteriorating impervious structural membranes, inadequate foundation cutoffs, desiccation of clay fill, steep slopes vulnerable to sliding, blocky foundation rock susceptible to differential settlement, ineffective contact at adjoining structures and at abutments, pervious embankment strata, vulnerability to conditions during an earthquake.

c. An embankment may be most vulnerable at its interface with rock abutments. Several dam failures have occurred during initial impoundment. Settlement in rockfill dams can be significantly reduced if the rockfill is mechanically compacted. In some ways, a compacted earth core is superior to a concrete slab as the impervious element of a rockfill dam. If the core has sufficient plasticity, it can be flexible enough to sustain pressures without significant damage.

9-10. Liquefaction

Liquefaction can occur during earthquakes. Hydraulic fill dams are particularly susceptible to this type of damage. Liquefaction is a potential problem for any embankment which has continuous layers of soil with uniform gradation and of fine grain size. The Fort Peck Dam experienced a massive slide on the upstream side in 1938, which brought the hydraulic fill dam under suspicion. The investigation at the time focused blame on an incompetent foundation, but few hydraulic fills were built after the 1930's. Heavy compaction equipment became available in the 1940's, and the rolled embankment dam became the competitive alternative.

9-11. Concrete Deterioration

Chemical and physical factors can age concrete. Visible clues to the deterioration include: expansion, cracking of random pattern, gelatinous discharge, and chalky surfaces.

9-12. PICES Measurements on Other USACE Navigation and Flood Control Structures

a. Concrete navigation lock monoliths and miter lock gates. The Corps operates approximately 270 navigation lock chambers constructed of plain or reinforced concrete. PICES surveys may be required over many of these structures to monitor potential problem areas. The frequency of these periodic surveys will be highly variable. Likewise, not all structural components of a lock complex (e.g., wall/monoliths, wing walls, gates, dam) may need to be monitored. These measurements are related to a list of 10 functional distresses. Observations for distresses in

miter lock gates may include one or more of the following:

Top anchorage movement
Elevation change
Miter offset
Bearing gaps
Downstream movement
Cracks
Leaks/boils
Dents
Abnormal noise or vibration during gate operation
Corrosion

b. Sheet pile structures. PICES observations for distresses in sheet pile structures may include one or more of the following:

Misalignment
Corrosion
Settlement
Cavity formation
Interlock separation
Holes
Dents
Cracks

c. Rubble breakwaters and jetties. Any number of measurements may be needed to monitor the condition of breakwater and jetties. These may involve either conventional surveying or hydrographic methods. Typical observations include measurements for the seaside and leeside slopes and crest in each of the following categories:

Seaside/Leeside Slope:	Crest/Cap:
Armor loss	Breaching
Armor quality defects	Armor loss
Lack of armor contact/interlock	
Core exposure/loss	Core exposure/loss
Slope defects	

9-13. Deformations in Large Structures

Deformation of large structures (e.g., dams) is mainly caused in part by the reservoir level. Other factors also play a role in the deformation of the structure, including temperature, self-weight of the material from which the dam is constructed, and earth pressure. A monitoring system should therefore include regular measurements of the reservoir level and temperature and pressure data.

a. Reservoir level measurement. Reservoir levels today should be measured with pressure balances. Double checking the measurements must be done and can be facilitated by installing a manometer on either an existing or new pipe connected to the reservoir. The measurement range should extend at least as far as the dam crest, thereby allowing observation and judgment of the flood risk and assessment of peak inflows.

b. Temperature measurement. Temperature measurement is required to determine the impact of temperature variations on the structure itself, as well as whether precipitation consists of rain or snow and if applicable, whether the snow melt period has begun. Temperature measurement should be done at least daily. Double checking is not necessary, as other methods can be used if failure takes place. The thermometers should be placed at various locations within the dam, either embedded in the structure itself or within drillholes. Redundancy should be provided for by using a greater number of thermometers than otherwise would be required.

c. Precipitation measurement. Precipitation measurement should be done by using a precipitation gauge. Daily readings are recommended. The gauging station does not need to be located at the dam site, but should not be too far away so as to not be representative of the precipitation level at the structure itself. Redundancy is not necessary for precipitation measurement as gauges located further away can often be used when the closer one fails. Every large structure has some form of seepage through the structure itself or its foundation, even if it has a grout curtain. In concrete dams, seepage typically is small and limited to permeable areas of the concrete, joints, and contact between rock and concrete. Any abnormal seepage is an immediate warning that something may be wrong with the structure or foundation. Seepage flows cause uplift pressure - pressure which must be monitored in view of its critical impact on the overall stability of the structure. In embankment (i.e., earthen) dams, seepage flow through the structure itself is similar to that observed in its foundation as the material from which both are made are pervious. Seepage flows not only cause uplift pressure in these structures, but also pore-water pressure. The pattern of seepage and water pressures on the structure (especially on the foundation and impervious core) has a significant impact on the behavior of the dam. Therefore, seepage is critical and must be carefully monitored as any abnormal rate may indicate a condition which is a serious threat to the overall stability and safety of the structure.

d. Seepage rate. The total seepage rate is the seepage at the face of the structure taken as a whole. As a whole, such rates can be assessed as to whether they are

"normal." Seepage rate can be measured volumetrically by using a calibrated container and a stopwatch or a gauging weir or flume. Such methods are straightforward, simple, and reliable; therefore redundancy is not necessary. Partial seepage rates are taken in isolated zones of the structure found to be representative for the area examined. Such rates should be monitored periodically. In the course of monitoring seepage, if an abnormality (i.e., change in normal seepage rate) is detected, the critical zone and cause of the seepage is easier to identify.

e. Chemical property analysis. If the structure is constructed of soluble or easily erodible material, the seepage should be monitored for turbidity and chemical content also. Doing so will permit the assessment of the overall stability of the embankment and foundation materials.

f. Pore-water pressure measurement. Structures usually are designed with specific pore-water pressure values that should not be exceeded. Pressure cells typically are designed or built into the structure themselves to measure pore-water pressure. The linking together of several cells forms a profile for the structure. The greater the number of measurement profiles and number of cells per profile, the more useful the data obtained will be. Even though pressure cells can be installed in structures themselves, rehabilitation of existing ones is not always practicable. Where pressure cells cannot be used to monitor pore-water pressure, the phreatic line in selected points will be monitored. Standpipe piezometers mounted in the embankment at several cross sections should be used to monitor the phreatic line.

g. Uplift pressure. Seepage underneath a structure causes uplift pressure which can severely alter the stabilizing effect of the structure's self weight. Uplift pressure can be reasonably controlled by a grout curtain and drainage holes, but uplift pressure and the physical effectiveness of these control measures should be carefully monitored. Piezometers connected to a manometer are a reliable means to measure the uplift pressure in cross-sections and several points on the upstream and downstream face of the structure.

h. Discharge measurement. If the foundation is being drained, drainage discharge should be monitored by either volumetric gauging or gauging weir. Such methods are sufficiently reliable not to require redundancy. Any change in flow rate may be indicative of clogging in the drainage system. If possible, the discharge of any spring, rivers, streams, flood control structure, etc. downstream of the structure should be monitored to gauge the effect of any discharge on them. Any variations in discharge may be indicative of a seepage problem.

9-14. Current Deformation Methods, Equipment, and Analysis Techniques

This section describes the latest technology used for performing both relative and absolute deformation surveys. Many of the procedures and methods contained herein require extensive geodetic survey background, and are applicable only to large, high head dams, or on structures with unique problems. They should not be employed on smaller structures or earthen embankments.

a. Deformation monitoring, analysis, and prediction are of a major and ever-growing concern in practically all fields of engineering and geoscience. Safety, economical design of man-made structures, efficient functioning and fitting of structural elements, environmental protection, and development of mitigative measures in the case of natural disasters (land slides, earthquakes, liquefaction of earth dams, etc.) require a good understanding of causes (loads) and the mechanism of deformation which can be achieved only through the proper monitoring and analysis of deformable bodies.

b. The development of new methods and techniques for the monitoring and analysis of deformations and the development of methods for the optimal modeling and prediction of deformations have been the subject of intensive studies by many professional groups at national and international levels. Within the most active international organizations which are involved in deformation studies one should list:

- International Federation of Surveyors (FIG) with its Study Group 6C which has significantly contributed to the recent development of new methods for the design and geometrical analysis of integrated deformation surveys and new concepts for global integrated analyses and modeling of deformations;

- International Commission on Large Dams (ICOLD) with its Committee on Monitoring of Dams and their Foundations;

- International Association of Geodesy (IAG) with the very active Commission on Recent Crustal Movements which frequently organizes international and regional symposia concerning geodynamics, tectonic plate movements, and modeling of regional earth crust deformation;

- International Society for Mine Surveying (ISM) with their very active Commission 4 on Ground Subsidence and Surface Protection in mining areas;

- International Society for Rock Mechanics (ISRM) with their overall interest in rock stability and ground control; and

- International Association of Hydrological Sciences (IAHS) which organizes international symposia (e.g., Venice 1984, Houston 1991) on ground subsidence due to the withdrawal of underground liquids (water, oil, etc.).

c. The FIG Study Group 6C has been one of the most, if not the most, active international groups dealing with practically all aspects of deformation monitoring and analysis. Since 1975, the FIG Study Group 6C has organized six international symposia with the last symposium held in Hanover in 1992. Although the activity of FIG in the development of new monitoring techniques is biased, of course, toward geodetic surveying techniques, its activity in the design and analysis of deformation surveys is more objective than that of any other professional group. In 1978, an ad hoc Committee on Deformation Analysis was formed to deal with and clarify various approaches and schools of thinking regarding the geometrical analysis of deformation surveys including the identification of unstable reference points. The work of the Committee has resulted in the development of new concepts for integrated monitoring systems (integration of geodetic and geotechnical measurements) and for generalized global analyses of integrated deformation surveys.

d. Most of the activities and studies of other associations and organizations focus, of course, on direct applications to their particular deformation problems. Although the accuracy and sensitivity criteria for the determination of deformation may considerably differ between various applications, the basic principles of the design of monitoring schemes and their geometrical analysis remain the same. For example, a study on the stability of magnets in a nuclear accelerator may require determination of relative displacements with an accuracy of 0.05 mm while a settlement study of a rock-fill dam may require only 10 mm or larger. In both cases, although the monitoring techniques and instrumentation may differ, one may show that the same basic methodology in the designing and analysis of the deformation measurements may be utilized. This fact has not yet been fully understood by most of the above-listed international research groups. There is a general lack of communication and work coordination.

The studies of various professional groups, not only at the international but also at the national level of individual countries, overlap resulting in the duplication of efforts in discovering methods and techniques which have already been well known to other study groups. For example, in the United States there is very little communication between the U.S. Bureau of Reclamation, U.S. Army Corps of Engineers, U.S. Commission on Large Dams, and the American Congress on Surveying and Mapping. All four organizations are involved separately in studies on deformations of engineering structures and in the development of guidelines for deformation monitoring. Even within the same organization or institution, one may find examples of two different professional groups, for instance, geotechnical and structural engineers, who may work on the same deformable object but do not exchange information on their methods and the results of their analyses.

e. According to the most recent information obtained from the U.S. Committee on Large Dams, USACE is in charge of 475 large dams out of a total of 5,469 large dams maintained currently in the United States. An additional 49 large dams are currently under construction.

9-15. Review of Monitoring Techniques and Instrumentation

a. The monitoring schemes include:

- Monitoring of deformations, i.e., determination of the geometrical change in shape and dimensions and rigid body translations and rotations (absolute and/or relative) of the monitored object; and

- Monitoring of acting forces (loads) and internal stresses which can either be measured directly or derived from measurements of temperature, pore water pressure, water level, seepage, etc.

b. In addition, laboratory and/or in situ determinations of physical properties of the deformable material (e.g., moduli of elasticity, tensile strength, creep parameters, porosity, etc.) are necessary for a proper design and evaluation of the behavior of the monitored structure. In seismically active areas, the monitoring schemes must include special instrumentation for measuring vibrations.

c. This section reviews only the techniques used in monitoring the deformations although all the other above-mentioned components of the monitoring schemes play an

equally important role in the analysis and interpretation of deformations as discussed below.

d. The measuring techniques and instrumentation for geometrical monitoring of deformations have traditionally been categorized into two groups according to the two main groups of professionals who use the techniques:

(1) Geodetic surveys which include conventional terrestrial, photogrammetric, satellite positioning, and some special techniques (interferometry, hydrostatic leveling, alignment, and other), and,

(2) Geotechnical/structural measurements of local deformations using tiltmeters, strainmeters, extensometers, joint meters, plumb lines, etc.

e. Each type of measurement has its own advantages and drawbacks. Geodetic surveys, through a network of points interconnected by angle and/or distance measurements, usually supply a sufficient redundancy of observations for the statistical evaluation of their quality and for a detection of errors. They give global information on the behavior of the deformable object while the geotechnical measurements give very localized and, very frequently, locally disturbed information without any check unless compared with some other independent measurements. On the other hand, geotechnical instruments are easier to adapt for automatic and continuous monitoring than conventional geodetic instruments. Conventional terrestrial surveys are labor intensive and require skilful observers, while geotechnical instruments, once installed, require only infrequent checks on their performance. Geodetic surveys have traditionally been used mainly for determining the absolute displacements of selected points on the surface of the object with respect to some reference points that are assumed to be stable. Geotechnical measurements have traditionally been used mainly for relative deformation measurements within the deformable object and its surroundings. However, with the technological progress of the last few years, the differences between the two techniques and their main applications are not as obvious as 20 years ago. For example, inverted plumb lines and borehole extensometers, if anchored deeply enough in bedrock below the deformation zone, may serve the same way as, or even better than, geodetic surveys for determining the absolute displacements of the object points. Geodetic surveys with optical and electromagnetic instruments (including satellite techniques) are always contaminated by atmospheric (tropospheric and ionospheric) refraction which limits their positioning accuracy to about ± 1 ppm to ± 2 ppm (at the standard deviation level) of the distance. So, for instance, with the

average distance between the object and reference points of 500 m, the absolute displacements of the object points cannot be determined with an accuracy better than about ± 2 mm at the 95 percent probability level. In some cases this accuracy is not adequate. On the other hand, precision electro-optical geodetic instruments for electronic distance measurements (EDM), with their accuracies of ± 0.3 mm over short distances, may serve as extensometers in relative deformation surveys. Similarly, geodetic leveling, with an achievable accuracy of better than ± 0.1 mm over distances of 20 m may provide better accuracy for the tilt determination (equivalent to ± 1 second of arc) than any local measurements with electronic tiltmeters. New developments in three-dimensional coordinating systems with electronic theodolites may provide relative positioning in almost real time to an accuracy of ± 0.05 mm over distances of several meters. The same applies to new developments in photogrammetric measurements with the solid state cameras (CCD sensors). The satellite GPS, which, if properly handled, offers a few millimeters accuracy in differential positioning over several kilometers, is replacing conventional terrestrial surveys in many deformation studies and, particularly, in establishing the reference networks.

f. A full review of all the instruments and techniques available for deformation monitoring would far exceed the scope of this manual. In view of the available references listed in Appendix A, the material below is limited only to some comments on the achievable accuracy of the basic types of instruments which include electronic instrumentation for distance and angle measurements, surveying robots, measurements of tilt and inclination, changes in distances and strain, photogrammetric methods, and the satellite GPS. It is assumed that the users of this manual possess the basic understanding of geodetic and geotechnical measuring techniques and do not require explanations of what is a theodolite, or a hydrostatic level, or a tiltmeter. Additional information can be found in the references in Appendix A.

9-16. Electronic Distance and Angle Measurements

a. Electronic theodolites. Over the last two decades, the technological progress in angle measurements has been mainly in the automation of the readout systems of the horizontal and vertical circles of the theodolites. The optical readout systems have been replaced by various, mainly photo-electronic, scanning systems of coded circles with an automatic digital display and transfer of the readout to electronic data collectors or computers. Either decimal units (gons) or traditional sexagesimal units of

degrees, minutes, and seconds of arc may be selected for the readout. The sexagesimal system of angular units, which is still commonly accepted in North America, is used throughout this section. The relationship between the two systems is that $360° = 400$ gons.

(1) As far as accuracy is concerned, electronic theodolites have not brought any drastic improvements in comparison with precision optical theodolites. Some of the precision electronic theodolites, such as the Kern E2 (discontinued production), Leica (Wild) T2002 and T3000, and a few others, are equipped with microprocessor controlled biaxial sensors (electronic tiltmeters) which can sense the inclination (misleveling) of the theodolite to an accuracy better than 0.5" and automatically correct not only vertical but also horizontal direction readouts. In optical theodolites in which the inclination is controlled only by a spirit level, errors of several seconds of arc in horizontal directions could be produced when observing along steeply inclined lines of sight. Therefore, when selecting an electronic theodolite for precision surveys, one should always choose one with the biaxial leveling compensator.

(2) Human errors of pointing the telescope to the target, centering errors, and environmental influences are the main factors limiting the achievable accuracy. The environmental influence of atmospheric refraction is a particular danger to any optical measurements. The gradient of air temperature, dT/dx, in the direction perpendicular to the line of sight is the main parameter of refraction. Assuming that a uniform temperature gradient persists over the whole length S of the line of sight, the refraction error e_{ref} of the observed direction may be approximately expressed in seconds of arc by:

$$e_{ref} = 8"(SP/T^2)dT/dx \qquad (9-1)$$

where P is the barometric pressure in millibars and T is the temperature in Kelvin (T = 273.15 + t°C). If a gradient of only 0.1°C/m persists over a distance of 500 m at P = 1000 mb and t = 27°C, it will cause a directional error of 4.4", which is equivalent to a 12-mm positional error of the target. One should always avoid measurements close to any surface that may have a different temperature than the surrounding air (walls of structures or soil exposed to the sun's radiation, walls of deep tunnels, etc.). If any suspicion of a refraction influence arises, the surveys should be repeated in different environmental conditions in order to randomize its effect.

(3) Generally, with well designed targets and proper methodology, an accuracy (standard deviation) better than

1" in angle measurements can be achieved with precision electronic theodolites if three to five sets of observations are taken in two positions (direct and reverse) of the telescope. The requirement of two positions must always be obeyed in order to eliminate errors caused by mechanical misalignment of the theodolite's axial system. This applies to both the old and most of the up-to-date theodolites even if the manufacturer claims that the errors are taken care of automatically.

b. Three-dimensional coordinating systems. Two or more electronic theodolites linked to a microcomputer create a three-dimensional (3-D) coordinating (positioning) system with on-line calculations of the coordinates. The systems are used for the highest precision positioning and deformation monitoring surveys over small areas. Leica (Wild) TMS and UPM400 (Geotronics, Sweden) are examples of such systems. If standard deviations of simultaneously measured horizontal and vertical angles do not exceed 1", then positions (x, y, z) of targets at distances up to 10 m away may be determined with the standard deviations smaller than 0.05 mm. Usually short invar rods of known length are included in the measuring scheme to provide scale for the calculation of coordinates.

c. Electronic distance measurements (EDM). Typically, short range (a few kilometers), electro-optical EDM instruments with visible or near infrared continuous radiation are used in engineering surveys, though some long range (up to 60 km) electro-optical or microwave instruments are also available.

(1) The accuracy (standard deviation) of EDM instruments may be expressed in a general form as:

$$\sigma = (a^2 + b^2S^2)^{0.5} \qquad (9-2)$$

where "a" contains errors of the phase measurement and calibration errors of the so-called zero correction (additive constant of the instrument and of the reflector), while the value of "b" represents a scale error due to the aforementioned uncertainties in the determination of the refractive index and errors in the calibration of the modulation frequency. Typically, the value of "a" ranges from 3 mm to 5 mm. In the highest precision EDM instruments, such as the Kern ME5000, Geomensor CR234 (Com-Rad, U.K.), and Tellurometer MA200 (Tellumat, U.K.), a = 0.2 mm to 0.5 mm thanks to a high modulation frequency and high resolution of the phase measurements in those instruments.

(2) One recently developed engineering survey instrument is Leica (Wild) DI2002 which offers a

standard deviation of 1 mm over short distances. Over distances longer than a few hundred meters, however, the prevailing error in all EDM instruments is due to the difficulty in determining the refractive index. Therefore, all EDM measurements must be corrected for the actual refractive index of air along the measured distance. An error of 1°C or an error of 3 mb in barometric pressure causes a 1 ppm (part per million or $1~mm \cdot km^{-1}$) error of the measured distance. An extremely careful measurement of the atmospheric conditions at several points along the optical path must be performed with well calibrated thermometers and barometers in order to achieve the 1-ppm accuracy. If the meteorological conditions are measured only at the instrument station (usual practice), then errors of a few parts per million may occur, particularly in diversified topographic conditions. In order to achieve the accuracy better than 1 ppm, one has to either measure the meteorological conditions every few hundred meters (200 m - 300 m) along the optical path or to use EDM instruments with a dual frequency radiation source.

(3) Only a few units of a dual frequency instrument (Terrameter LDM2 by Terra Technology) are available around the world. They are bulky and capricious in use but one may achieve with them a standard deviation of ±0.1 mm ±0.1 ppm. Due to a small demand, its production has been discontinued. Research in the development of new dual frequency instruments is in progress. In deformation measurements one may reduce somewhat the influence of refraction by "calibrating" the distance observations to the object targets by comparing the results with the "fixed" distances between stable stations of the reference network.

(4) Influence of relative humidity may be neglected when using common electro-optical EDM instruments in moderate climatic conditions. The negligence of humidity, however, may cause errors up to 2 ppm in extremely hot and humid conditions. Therefore, in the highest precision measurements, psychrometers with wet and dry thermometers should be used to determine the correction due to water vapor content. One should always use rigorous formulas to calculate the refractive index correction rather than diagrams or simplified calculation methods supplied by the manufacturers.

(5) All EDM instruments must be frequently calibrated for the zero correction and for scale (change in the modulation frequency). The zero correction usually significantly changes with time and may also be a function of the intensity of the reflected signal. In some older EDM instruments, the zero correction may demonstrate phase dependent cyclic changes. In engineering projects of high precision, the EDM instruments should be calibrated at least twice a year, or before and after each important project. The calibration must account for all combinations of EDM-reflector pairings since each reflector may also have a different additive constant correction. Additional errors, which are not included in the general error equation, may arise when reducing the results of the spatially measured distances to a reference plane depending on the accuracy of the reduction corrections.

(6) Recently, a few models of EDM instruments with a short pulse transmission and direct measurement of the propagation time have become available. These instruments, having a high energy transmitted signal, may be used without reflectors to measure short distances (up to 200 m) directly to walls or natural flat surfaces with an accuracy of about 10 m. Examples are the Pulsar 500 (Fennel, Germany) and the Leica (Wild) DIOR 3002.

d. Total stations and survey robots.

(1) Any electronic theodolite linked to an EDM instrument and to a computer creates a total surveying station which allows for a simultaneous measurement of the three basic positioning parameters, distance, horizontal direction, and vertical angle, from which relative horizontal and vertical positions of the observed points can be determined directly in the field. Several manufacturers of survey equipment produce integrated total stations in which the EDM and electronic angle measurement systems are incorporated into one compact instrument with common pointing optics. Different models of total stations vary in accuracy, range, sophistication of the automatic data collection, and possibilities for on-line data processing. One of the most recommended total stations for precision engineering surveys is the Leica (Wild) TC2002 which combines the precision of the aforementioned electronic theodolite, Leica (Wild) T2002, with the precision EDM instrument, Leica (Wild) DI2002, into one instrument with a coaxial optics for both the angle and distance measurements.

(2) For continuous or frequent monitoring of deformations, fully automatic monitoring systems based on computerized and motorized total stations have recently been developed. The first system was Georobot. The recent advanced systems include, for example, the Geodimeter 140 SMS (Slope Monitoring System) and the Leica (Wild) APS and Georobot III systems based on the motorized TM 3000 series of Leica (Wild) electronic theodolites linked together with any Leica (Wild) DI series of EDM. These can be programmed for sequential self-pointing to a set of prism targets at predetermined

time intervals, can measure distances and horizontal and vertical angles, and can transmit the data to the office computer via a telemetry link. Similar systems are being developed by other manufacturers of surveying equipment. The robotic systems have found many applications, particularly in monitoring high walls in open pit mining and in slope stability studies. Generally, the accuracy of direction measurements with the self-pointing computerized theodolites is less accurate (about 3 sec) than the measurements with manual pointing.

9-17. Leveling and Trigonometric Height Measurements

a. The old method of geometrical leveling with horizontal lines of sight (using spirit or compensated levels) is still the most reliable and accurate, though slow, surveying method. With high magnification leveling instruments, equipped with the parallel glass plate micrometer and with invar graduated rods, a standard deviation smaller than 0.1 mm per setup may be achieved in height difference determination as long as the balanced lines of sight do not exceed 20 m. In leveling over long distances (with a number of instrument setups) with the lines of sight not exceeding 30 m, a standard deviation of 1 mm per km may be achieved in flat terrain. The aforementioned influences of atmospheric refraction and earth curvature are minimized by balancing the lines of sight between the forward and backward leveling rods. A dangerous accumulation of refraction error, up to 15 mm for each 100-m difference in elevation, may take place along moderately inclined long routes due to unequal heights of the forward and backward horizontal lines above the terrain.

b. The recently developed Leica (Wild) NA2000 and NA3000 digital automatic leveling systems with height and distance readout from encoded leveling rods have considerably increased the speed of leveling (by about 30 percent) and decreased the number of personnel needed on the survey crew. However, some users of the digital level NA3000 complain that its compensating system demonstrates systematic deviations in windy weather and, therefore, cannot be classified as a high precision level unless some improvements are introduced by the manufacturer.

c. High precision electronic theodolites and EDM equipment allow for the replacement of geodetic leveling with more economical trigonometric height measurements. Using precision electronic theodolites for vertical angle measurements and any short-range EDM instrument, one may achieve an accuracy better than 1 mm in height

difference determination between two targets 200 m apart. To minimize the atmospheric refraction effects, the measurements must be performed either reciprocally, with two theodolites simultaneously, or from an auxiliary station with equal distances to the two targets (similar methodology as in spirit leveling). The accuracy is practically independent of the height differences and, therefore, is especially more economical than conventional leveling in hilly terrain and in all situations where large height differences between survey stations are involved. Motorized trigonometric height traversing (reciprocal or with balanced lines of sight) with precision theodolites and with the lines of sight not exceeding 250 m can give a standard deviation smaller than 2 mm per km. With automatic data collection and on-line processing of the measurements, daily progress of up to 15 km may be achieved independent of the terrain configuration. The refraction error is still the major problem in further increasing the accuracy of leveling. Research in this area continues.

9-18. Use of GPS in Deformation Surveys

a. The satellite GPS offers several advantages over conventional terrestrial methods. Intervisibility between stations is unnecessary, thus allowing greater flexibility in the selection of station locations than in the terrestrial geodetic surveys. Measurements can be taken during night or day, under varying weather conditions, which makes GPS measurements economical. With the recently developed rapid static positioning techniques, the time for the measurements at each station is reduced to a few minutes. Reference EM 1110-1-1003.

b. Though already widely used in engineering and geoscience projects, GPS is still a new and not perfectly known technology from the point of view of its optimal use and understanding of the sources of errors. The accuracy of GPS is very often exaggerated by some authors who may not quite understand the difference between the short-term precision (repeatability) and actual accuracy of GPS.

c. The accuracy of GPS relative positioning depends on the distribution (positional geometry) of the observed satellites and on the quality of the observations. There are several sources of errors contaminating the GPS measurements. These errors can be categorized into:

(1) Signal propagation errors which include effects of tropospheric and ionospheric refraction,

(2) Receiver related errors which include multipath effects, variation in the antenna phase center, receiver

noise, bias in the coordinates of the station being held fixed in the data reduction process, etc., and

(3) Satellite related errors which include mainly orbital errors.

d. Different types of errors affect GPS relative positioning in different ways. Some of the errors may have a systematic effect on the measured baselines producing significant scale errors and rotations. Due to the changeable geometrical distribution of the satellites and the resulting changeable systematic effects of the observation errors, repeated GPS surveys for the purpose of monitoring deformations can also be significantly influenced (up to a few ppm) by scale and rotation errors which, if undetected, may contaminate the derived deformation parameters leading to a misinterpretation of the behavior of the deformable body. A particular attention to the systematic influences should be paid when a GPS network is established along the shore of a large body of water and measurements are performed in a hot and humid climate.

e. Experience with the use of GPS in various deformation studies indicates that, with the available technology (receivers) and the distribution of the satellites in 1990-1991, the accuracy of GPS relative positioning over areas of up to 50 km in diameter can be expressed in terms of the variance of the horizontal components of the GPS baselines, over a distance S, as

$$\sigma^2 = (3 \text{ mm})^2 + (10^{-6} \text{ S})^2$$

if the aforementioned systematic biases (rotations and change in scale of the network) are identified and eliminated through proper modeling at the stage of the deformation interpretation. The accuracies of vertical components of the baselines are, usually, 1.5 to 2.5 times less accurate than the horizontal components.

f. The solution for the systematic parameters may be obtained by either (1) combining the GPS surveys of some baselines (of a different orientation) with terrestrial surveys of a compatible or better accuracy or (2) establishing several points outside the deformable area (fiducial stations) which would serve as a "calibration network" or (3) combining (1) and (2). These aspects must be considered when designing GPS networks for any engineering project. In the first case, for all the terrestrial $\mathbf{l}_T$ observables and for GPS observables $\mathbf{l}_G$ one could write observation equations for epochs t_o and t_i in terms of the deformation model $\mathbf{Bc}$ (displacement function) in the form:

$$\mathbf{l}_T(t_i) + \mathbf{v}_T(t_i) = \mathbf{l}_T(t_o) + \mathbf{A}_T \mathbf{B}_l \mathbf{c},$$

for all i, and

$$\mathbf{l}_G(t_i) + \mathbf{v}_G(t_i) = \mathbf{l}_G(t_o) + \mathbf{A}_G \mathbf{B}_l \mathbf{c} + \mathbf{D}d\eta$$

for all i,   (9-3)

where $d\eta$ is the vector of changes in scale and rotation parameters between the epochs t_o and t_i, $\mathbf{B}_l$ is the matrix constructed by superimposing matrices $\mathbf{B}$ for all the surveyed points and all the epochs, and $\mathbf{A}$ is the design matrix relating observables to the deformation model. The elements of the vectors $\mathbf{c}$ and $d\eta$ are estimated using the least squares method and they are statistically tested for their significance.

g. The influence of systematic errors in measurements over short distances (up to a few hundred meters) is usually negligible and the horizontal components of the GPS baselines can be determined with standard deviations of 3 mm or even smaller. Recent improvements to the software for the GPS data processing allow for an almost real time determination of changes in the positions of GPS stations.

h. In 1991, USACE developed a fully automated system for high-precision deformation surveys with GPS. It was designed particularly for dam monitoring. In the continuous deformation monitoring system (CDMS), GPS antennas are located over points to be monitored on the structures. At least two other GPS antennas must be located over reference points that are considered stable. The GPS antennas are connected to computers using a telemetry link. A prototype system used 10-channel Trimble 4000SL and Trimvec postprocessing software. An operator can access the onsite computer network through a remote hook-up in the office. In 1989 the system was installed at the Dworshak Dam on the Clearwater River near Orofino, ID. The results show that CDMS can give accuracies of 3 mm both horizontally and vertically over a 300-m baseline. One has to be aware, however, that although GPS does not require the intervisibility between the observing stations it requires an unobstructed view to the satellites which limits the use of GPS only to reasonably open areas. One should also remember that there might be some yet undiscovered sources of errors (e.g., effects of high voltage power lines) in GPS measurements. GPS certainly revolutionizes the geodetic surveys but still more research on its optimal use and on sources of errors in deformation surveys is needed. See also EM 1110-1-1003 for details on this system.

9-19. Photogrammetric Techniques

a. If an object is photographed from two or more survey points of known relative positions (known coordinates) with a known relative orientation of the camera(s), relative positions of any identifiable object points can be determined from the geometrical relationship between the intersecting optical rays which connect the image and object points. If the relative positions and orientation of the camera are unknown, some control points on the object must be first positioned using other surveying techniques. Aerial photogrammetry has been extensively used in determining ground movements in ground subsidence studies in mining areas, and terrestrial photogrammetry has been used in monitoring of engineering structures. The main advantages of using photogrammetry are: the reduced time of field work; simultaneous provision of 3-D coordinates; and, in principle, an unlimited number of points can be monitored. The accuracy of photogrammetric point determination has been much improved in the past decade, which makes it attractive for high precision deformation measurements.

b. Special cameras with minimized optical and film distortions must be used in precision photogrammetry. Cameras combined with theodolites (phototheodolites), for instance the Wild P-30 model, or stereocameras (two cameras mounted on a bar of known length) have found many applications in terrestrial engineering surveys including mapping and volume determination of underground excavations and profiling of tunnels. The accuracy of photogrammetric positioning with special cameras depends mainly on the accuracy of the determination of the image coordinates and the scale of the photographs. The image coordinates may, typically, be determined with an accuracy of about 10 μm, though 3 μm is achievable. The photo scale may be approximately expressed as f/s, where f is the focal length of the objective lens and s is the distance of the camera from the object. Using a camera with, for instance, f = 100 mm at a distance s = 100 m, with the accuracy of the image coordinates of 10 μm, the coordinates of the object points can be determined with the accuracy of 10 mm. Special large-format cameras with long focal length are used in close-range industrial applications of high precision. For instance, the model CRC-1 (Geodetic Services, Inc., U.S.A.) camera with f = 240 mm, can give sub-millimeter accuracy in "mapping" objects up to a few tens of meters away. Recently, solid-state cameras with charge couple device (CCD) sensors have become available for close-range photogrammetry in static as well as in dynamic applications. With the new developments in CCD cameras and digital image processing techniques,

continuous monitoring with real-time photogrammetry becomes possible. Further development in this area is in progress.

9-20. Alignment Measurements

a. Alignment surveys cover an extremely wide spectrum of engineering applications from the tooling industry, through measurements of amplitude of vibrations of engineering structures, to deformation monitoring of nuclear accelerometers several kilometers long. Each application may require different specialized equipment.

b. The methods used in practice may be classified according to the method of establishing the reference line, that is: (1) mechanical methods in which stretched wire (steel, nylon, etc.) establishes the reference line, (2) direct optical method (called also collimation method), in which the optical line of sight or a laser beam "marks" the line, and (3) diffraction method in which the reference line is created by projecting a pattern of diffraction slits.

c. All the above methods except mechanical are affected by atmospheric refraction, as expressed by Equation 9-1. Therefore, in measurements requiring high accuracy, the alignment must be repeated several times in different environmental conditions.

d. The mechanical methods with tensioned wires as the reference lines have found many applications including dam deformation surveys. This is due to their simplicity, high accuracy, and easy adaptation to continuous monitoring of structural deformations using inductive sensors over distances up to a few hundred meters. Accuracies of 0.1 mm are achievable.

e. The direct optical method utilizes either an optical telescope and movable targets with micrometric sliding devices or a collimated (projected through the telescope) laser beam and movable photocentering targets. Besides the aforementioned influence of atmospheric refraction, pointing and focusing are the main sources of error when using optical telescopes. The pointing error with properly designed targets varies from 15"/M at night in calm atmospheric conditions to 60"/M in daylight with average turbulent conditions, where M is the magnification of the telescope.

f. Special aligning telescopes with large magnification (up to 100x) are available from, among others, Fennel-Cassell (Germany) and Zeiss-Jena (Germany). Aligning telescopes for the tooling industry and machinery alignment are available in North America from

Cubic Precision. When the optical line of sight is replaced by a collimated laser beam, then the accuracy of pointing may be considerably improved if special self-centering laser detectors, with a time integration of the laser beam energy, are used. The use of laser allows for automation of the alignment procedure and for continuous data acquisition. When using the laser beam directly as the reference line, however, attention must be paid to the stability of the laser cavity. A directional drift of the laser beam as high as 4"/°C may occur due to thermal effects on the laser cavity. This effect is decreased by a factor of M when projecting the laser through a telescope.

g. In diffraction alignment methods, a pinhole source of monochromatic (laser) light, the center of a plate with diffraction slits, and the center of an optical or photoelectric sensor are the three basic points of the alignment line. If two of the three points are fixed in their position, then the third may be aligned by centering the reticle on the interference pattern created by the diffraction grating. It should be pointed out that movements of the laser and of its output do not influence the accuracy of this method of alignment because the laser serves only as a source of monochromatic light placed behind the pinhole and not as the reference line. Therefore, any kind of laser may be employed in this method, even the simplest and least expensive ones, as long as the output power requirements are satisfied. Various patterns of diffraction slits are used in practice. The highest accuracy and the longest range are obtained with the so-called Fresnel zone plates which act as focusing lenses. For instance, rectangular Fresnel zone plates with an electro-optical centering device were used in alignment and deformation measurements of a 3-km-long nuclear accelerator giving relative accuracy (in a vacuum) of 10^{-7} of the distance. In the open atmosphere, the thermal turbulence of air seems to have a smaller effect when using the Fresnel zone plates than in the case of direct optical alignment. The laser diffraction alignment methods have successfully been applied in monitoring both straight and curved (arch) dams using self-centering targets with automatic data recording.

9-21. Measurement of Extension (Change in Distance) and Strain

a. Types of extensometers. Various types of instruments, mainly mechanical and electro-mechanical, are used to measure changes in distance in order to determine compaction or upheaval of soil, convergence of walls in engineering structures and underground excavations, strain in rocks and in man-made materials, separation between rock layers around driven tunnels, slope stability, and movements of structures with respect to the foundation rocks. Depending on its particular application, the same instrument may be named an extensometer, strainmeter, convergencemeter, or fissuremeter.

(1) The various instruments differ from each other by the method of linking together the points between which the change in the distance is to be determined and the kind of sensor employed to measure the change. The links in most instruments are mechanical, such as wires, rods, or tubes. The sensors usually are mechanical, such as calipers or dial gauges. In order to adapt them to automatic and continuous data recording, electric transducers can be employed using, for instance, linear potentiometers, differential transformers, and self-inductance resonant circuits. In general, when choosing the kind of transducer for automatic data acquisition, one should consult with an electronics specialist on which kind would best suit the purpose of the measurements in the given environmental conditions.

(2) One should point out that the precision EDM instruments, as described earlier with their accuracy of 0.3 mm over short distances, may also be used as extensometers particularly when the distances involved are several tens of meters long.

(3) If an extensometer is installed in the material with a homogeneous strain field, then the measured change Δl of the distance l gives directly the strain component $\varepsilon = \Delta l/l$ in the direction of the measurements. To determine the total strain tensor in a plane (two normal strains and one shearing), a minimum of three extensometers must be installed in three different directions.

b. Wire and tape extensometers. Maintaining a constant tension throughout the use of the wire or tape extensometer is very important. In some portable extensometers, the constant tensioning weight has been replaced by precision tensioning springs. One should be careful because there are several models of spring tensioned extensometers on the market which do not provide any means of tension calibration. As the spring ages, these instruments may indicate false expansion results unless they are carefully calibrated on a baseline of constant length, before and after each measuring campaign.

(1) Among the most precise wire extensometers are the Kern Distometer (discontinued production) and the CERN Distinvar (Switzerland). Both instruments use invar wires and special constant tensioning devices which, if properly calibrated and used, can give accuracies of

0.05 mm or better in measurements of changes of distances over lengths from about 1 m to about 20 m. Invar is a capricious alloy and must be handled very carefully to avoid sudden changes in the length of the wire. When only small changes in temperature are expected or a smaller precision (0.1 mm to 1 mm) is required, then steel wires or steel tapes are more comfortable to use.

(2) Special high precision strainmeters of a short length (up to a few decimeters) are available for strain measurements in structural material and in homogeneous rocks. An example is a vibrating wire strain gauge available from Rocktest (formerly Irad Gage). The instrument employs a 150-mm steel wire in which the changeable resonant frequency is measured. An accuracy of one microstrain (10^{-6}) is claimed in the strain measurements which corresponds to 0.15 μm relative displacements of points over a distance of 150 mm.

c. Rod, tube, and torpedo extensometers. Steel, invar, aluminum, or fiberglass rods of various lengths, together with sensors of their movements, may be used depending on the application. Multiple point measurements in boreholes or in trenches may be made using either a parallel arrangement of rods anchored at different distances from the sensing head, or a string (in series) arrangement with intermediate sensors of the relative movements of the rods.

(1) A typical accuracy of 0.1 mm to 0.5 mm may be achieved up to a total length of 200 m (usually in segments of 3 m to 6 m). The actual accuracy depends on the temperature corrections and on the quality of the installation of the extensometer. When installing rods in plastic conduit (usually when installing in boreholes), the friction between the rod and the conduit may significantly distort the extensometer indications if the length of the extensometer exceeds a few tens of meters. The dial indicator readout may be replaced by potentiometric or other transducers with digital readout systems. Telescopic tubes may replace rods in some simple applications, for instance, in measurements of convergence between the roof and floor of openings in underground mining.

(2) Several models of torpedo borehole extensometers and sliding micrometers are available from different companies producing geotechnical instrumentation. For example, Extensofor (Telemac, France) consists of a 28-mm-diameter torpedo 1.55 m long with an inductance sensor at each end. Reference rings on the casing are spaced within the length of the torpedo. The sensors and reference rings form the inductance oscillating circuits. The torpedo is lowered in the borehole and stopped

between the successive rings recording changes in distances between the pairs of rings with a claimed accuracy of 0.1 mm. Boreholes up to several hundreds of meters long can be scanned.

d. Interferometric measurements of linear displacements. Various kinds of interferometers using lasers as a source of monochromatic radiation are becoming common tools in precision displacement measurements. A linear resolution of 0.01 μm, or even better, is achievable. One has to remember, however, that interferometric distance measurements are affected by atmospheric refractivity in the same way as all EDM systems. Therefore, even if temperature and barometric pressure corrections are applied, the practical accuracy limit is about 10^{-6} S (equivalent to 1 μm per meter). Thermal turbulence of air limits the range of interferometric measurements in the open atmosphere to about 60 m. The Hewlett Packard Model 5526B laser interferometer has found many industrial and laboratory applications in the measurement of small displacements and the calibration of surveying instruments.

e. Use of optical fibre sensors. A new interesting development in the measurements of extensions and changes in crack width employs a fully automatic extensometer which utilizes the principle of electro-optical distance measurements within fibre optic conduits. The changes in length of the fibre optic sensors are sensed electro-optically and they are computer controlled .

9-22. Tilt and Inclination Measurements

a. Methods of tilt measurements.

(1) The measurement of tilt is usually understood as the determination of a deviation from the horizontal plane, while inclination is interpreted as a deviation from the vertical. Thus the same instrument that measures tilt at a point can be called either a tiltmeter or an inclinometer, depending on the interpretation of the results.

(2) As discussed previously, geodetic leveling techniques can achieve an accuracy of 0.1 mm over a distance of 20 m, which would be equivalent to about 1.0" of angular tilt. This accuracy is more than sufficient in most engineering deformation measurements. Whenever a higher accuracy or continuous or very frequent collection of information on the tilt changes is necessary, however, various in situ instruments are used, such as (a) engineering tiltmeters and inclinometers; (b) suspended and inverted plumb lines; and (c) hydrostatic levels. In addition, some other specialized instruments such as

mercury/laser levels have been developed but are not commonly used in practice and, therefore, are not reviewed in this section.

b. Tiltmeters and inclinometers. There are many reasonably priced models of various liquid, electrolytic, vibrating wire, and pendulum type tiltmeters that satisfy most of the needs of engineering surveys. Particularly popular are servo-accelerometer tiltmeters with a small horizontal pendulum. They offer ruggedness, durability, and low temperature operation. The output signal (volts) is proportional to the sine of the angle of tilt. The typical output voltage range for tiltmeters is ±5 V, which corresponds to the maximum range of the tilt. Thus the angular resolution depends on the tilt range of the selected model of tiltmeter and the resolution of the voltmeter (typically 1 mV). There are many factors affecting the accuracy of tilt sensing. A temperature change produces dimensional changes of the mechanical components, changes in the viscosity of the liquid in the electrolytic tiltmeters, and changes of the damping oil in the pendulum tiltmeters. Drifts of tilt indications and fluctuations of the readout may also occur. Therefore, thorough testing and calibration are required even when the accuracy requirement is not very high.

(1) Tiltmeters have a wide range of applications. A series of tiltmeters if arranged along a terrain profile may replace geodetic leveling in the determination of ground subsidence. Similarly, deformation profiles of tall structures may be determined by placing a series of tiltmeters at different levels of the structure.

(2) In geomechanical engineering, the most popular application of tiltmeters is in slope stability studies and in monitoring embankment dams using the torpedo (scanning) type borehole inclinometers (usually the servo-accelerometer type tiltmeters). The biaxial inclinometers are used to scan boreholes drilled to the depth of an expected stable strata in the slope. By lowering the inclinometer on a cable with marked intervals and taking readings of the inclinometer at those intervals, a full profile of the borehole and its changes may be determined through repeated surveys. Usually the servo-accelerometer inclinometers are used with various ranges of inclination measurements, for instance, ±6°, ±54°, or even ±90°. If a 40-m-deep borehole is measured every 50 cm with an inclinometer of only 100" accuracy, then the linear lateral displacement of the collar of the borehole could be determined with an accuracy of 2 mm. A fully automatic (computerized) borehole scanning inclinometer system with a telemetric data acquisition has been designed at the University of New Brunswick for monitoring slope stability at the Syncrude Canada tar sands mining operation.

c. Suspended and inverted plumb lines. Two kinds of mechanical plumbing are used in controlling the stability of vertical structures: (1) suspended plumb lines and (2) floating plumb lines, also called inverted or reversed plumb lines. Inverted plumb lines have an advantage over suspended plumb lines in the possibility of monitoring absolute displacements of structures with respect to deeply anchored points in the foundation rocks which may be considered as stable. In the case of power dams, the depth of the anchors must be 50 m or even more below the foundation in order to obtain absolute displacements of the surface points. If invar wire is used for the inverted plumb line, vertical movements of the investigated structure with respect to the bedrock can also be determined. Caution must be used in installing plumb lines. If the plumb line is installed outside the dam, a vertical pipe of a proper inner diameter should be used to protect the wire from the wind. The main concern with floating plumb lines is to ensure verticality of the boreholes so that the wire of the plumb line has freedom of motion. The tank containing the float is generally filled with water to which some antifreeze can be added. The volume of the float should be such as to exert sufficient tension on the wire. It should also be noted, however, that in a float tank thermal convection displacements may easily develop in consequence of thermal gradients which may affect measurements to a considerable extent. Hence in some cases, the whole tank should be thermally insulated.

(1) Several types of recording devices that measure displacements of structural points with respect to the vertical plumb lines are produced by different companies. The simplest are mechanical or electromechanical micrometers. With these, the plumb wire can be positioned with respect to reference lines of a recording (coordinating) table to an accuracy of ±0.1 mm or better. Travelling microscopes may give the same accuracy. Automatic sensing and recording is possible, for instance, with a Telecoordinator (Huggenberger, Switzerland) and with a Telependulum (Telemac, France). An interesting Automated Vision System has been developed by Spectron Engineering. The system uses CCD video cameras to image the plumb line with a resolution of about 3 μm over a range of 75 mm. Several plumb lines at the Glen Canyon dam and at the Monticello dam in California have used this system.

(2) Two sources of error which may sometimes be underestimated by users are: the influence of air currents

and the spiral shape of wires. To reduce the influence of the air pressure, the plumb line should be protected within a pipe (e.g., a PVC tube) with openings only at the reading tables.

d. Optical plummets. Several surveying instrument companies produce high precision optical plummets: Leica (Wild) ZL (zenith) and NL (nadir) plummets which offer the accuracy of 1/200,000. Both can be equipped with laser. The atmospheric refraction is the major source of errors.

e. Hydrostatic leveling. If two connected containers are partially filled with a liquid, then the heights h_1 and h_2 of the liquid in the containers are related through the hydrostatic equation

$$h_1 + P_1 / (g_1 \, r_1) = h_2 + P_2 / (g_2 \, r_2) = const.$$

where P is the barometric pressure, g is gravity, and r is the density of the liquid which is a function of temperature.

(1) The above relationship has been employed in hydrostatic leveling. The ELWAAG 001 (Bayernwerke, Germany) is a fully automatic instrument with a travelling (by means of an electric stepping motor) sensor pin which closes the electric circuit upon touching the surface of the liquid.

(2) Hydrostatic leveling is frequently used in the form of a network of permanently installed instruments filled with a liquid and connected by hose-pipes to monitor change in height differences of large structures. The height differences of the liquid levels are automatically recorded. The accuracy ranges from 0.1 mm to 0.01 mm over a few tens of meters depending on the types of instruments. The main factor limiting the survey accuracy is the temperature effect. To reduce this effect the instrument must either be installed in a place with small temperature variations, or the temperature along the pipes must be measured and corrections applied, or a double liquid (e.g., water and mercury) is employed to derive the correction for this effect. For the highest accuracy, water of a constant temperature is pumped into the system just before taking the readings. The instruments with direct measurement of the liquid levels are limited in the vertical range by the height of the containers. This problem may be overcome if liquid pressures are measured instead of the changes in elevation of the water levels. Pneumatic pressure cells or pressure transducer cells may be used.

9-23. Concluding Remarks on Monitoring Techniques

This brief review of basic monitoring techniques indicates that, from the point of view of the achievable instrumental accuracy, the distinction between geodetic and geotechnical techniques does not apply any more. With the recent technological developments in both geodetic and geotechnical instrumentation, at a cost one may achieve almost any, practically needed, instrumental resolution and precision, full automation, and virtually real-time data processing. Remotely controlled telemetric data acquisition systems, working continuously for several months without recharging the batteries in temperatures down to -40°C, are available and their cost is reasonable. Thus, the array of different types of instruments available for deformation studies has significantly broadened within the last few years. This creates a new challenge for the designers of the monitoring surveys: what instruments to choose, where to locate them, and how to combine them into one integrated monitoring scheme in which the geodetic and geotechnical/structural measurements would optimally complement each other.

a. As far as the actual accuracy of deformation surveys is concerned, the main limiting factors are not the instrument precision but the environmental influences and ignorance of the users, namely:

- The aforementioned atmospheric refraction.

- Thermal influences affecting the mechanical, electronic, and optical components of the instruments (in any type of instrumentation) as well as the stability of survey stations.

- Local instability of the observation stations (improper monumentation of survey stations and improper installation of the in situ instrumentation).

- Lack of or improper calibration of the instruments.

- Lack of understanding by the users of the sources of errors and of the proper use of the collected observations.

b. The problem of calibration is very often underestimated in practice not only by the users but also by the manufacturers. In long-term measurements, the

instrument repeatability (precision) may be affected by aging of the electronic and mechanical components resulting in a drift of the instrument readout. Of particular concern are geotechnical instruments for which the users, in general, do not have sufficient facilities and adequate knowledge for their calibration. The permanently installed instruments are very often left in situ for several years without checking the quality of their performance.

c. The last aspect, the lack of understanding of the sources of errors affecting various types of measurements and the proper data handling is, perhaps, the most dangerous and, unfortunately, the frequent case in measurements of deformations in North America. The measurements, and particularly processing of the geodetic surveys, are usually in the hands of self-proclaimed "surveyors" at the technician level or even without any formal education. In this case, even the most technologically advanced instrumentation will not supply the expected information.

9-24. Design of Monitoring Schemes

a. When designing a monitoring network one has to remember that the main purpose of the monitoring surveys is:

- To check whether the behavior of the investigated object and its environment follows the predicted pattern so that any unpredicted deformations could be detected at an early stage.

- In the case of any abnormal behavior, to give an account, as accurately as possible, of the actual deformation status which could be used for the determination of the causative factors which trigger the deformation.

b. In the first case, the design of the monitoring scheme must include stations at the points where maximum deformations have been predicted plus a few observables at the points which, depending on previous experience, could signal any potential unpredictable behavior, particularly at the interface between the monitored structure and the surrounding material. The amount of the expected deformations may be predicted using either deterministic modeling (using, for instance, the finite or boundary element methods), or empirical (statistical) prediction models. Once any abnormal deformations are noticed, additional observables have to be added at the locations which would be indicated by the preliminary analysis of the monitoring surveys as being the most sensitive for the identification of causative factors. Some redundant monitoring instruments and points are absolutely necessary for checking the reliability of the measurements, especially in some critical parts of the structure. One should bear in mind that any monitoring instruments, even if they have been installed permanently, cannot rule out defects and failures. Thus, any monitoring system should be sufficiently redundant. By redundancy one means keeping parallel but separate sets of instruments and, in addition, facilities for evaluating data by double-checking, using alternative measurement methods. Examples would be the determination of relative displacements using alignment surveys versus displacements obtained from a geodetic monitoring network, the measurement of tilts with tiltmeters versus geodetic leveling, etc. Thus, a properly designed monitoring scheme should have a sufficient redundancy of measurements using different measuring techniques and such geometry of the scheme that self-checking, through geometrical closures of loops of measurements, would be possible. One should stress that a poorly designed monitoring survey is a waste of effort and money and may lead to a dangerous misinterpretation.

c. The accuracy (at the 95 percent probability level) of the monitoring measurements should be equal to at least 0.25 of the predicted value of the maximum deformations for the given span of time between the repeated measurements. However, once any abnormal deformations are noticed, there is no limit, other than economic, for the maximum possible accuracy required. The higher the accuracy of the measurements, the easier it will be to determine the mechanism of the unpredicted deformations. Thus, the monitoring schemes may require frequent updating and upgrading of the initial design over the duration of the monitoring project.

d. Generally, the design of a monitoring scheme includes, among many other aspects, the following tasks:

- Identification of the parameters to be observed.

- Selection of locations for the monitoring stations (both object and reference points if applicable).

- Determination (preanalysis) of the required accuracy and of the measuring range.

- Determination of the required frequency of repeated observations.

- Selection of the types of instruments and sensors to be used (various alternatives).

- Design of testing and calibration facilities.

- Design of the data management system.

- Preparation of a scenario for instrumentation failure (design of redundancy).

- Cost analysis and final decision on the selected monitoring scheme.

e. Since each deformable object may require different parameters to be observed, different instrumentation, different accuracies, and different frequencies of the observation, detailed specifications will significantly vary not only from one type of object to another, but also for the same type of object depending on the local surrounding conditions. Therefore, the following brief discussion on the first four tasks as listed above is very general and has been limited to typical conditions of concrete and embankment dams only.

9-25. Basic Considerations in Designing Monitoring Schemes for Large Dams

a. General deformation behavior of dams. Any dam is subjected to external and internal loads that cause deformation and permeability of the structure and its foundation. Deformation and seepage are clearly a function of such loads. Any sign of abnormal dam behavior could signal a threat to dam safety. In view of the difference between concrete and embankment dam behavior, a monitoring scheme cannot be organized in the same way for both types. In concrete dams, monitoring is essentially a matter of observing behavioral trends in both elastic and plastic deformation. The work consists of comparing measured deformation to the predicted normal behavior, assessed through analysis or some other method. In embankment dams on the other hand, permanent deformation trends should be closely monitored for any sign of abnormality.

b. Identification of parameters to be observed in concrete dams.

(1) Absolute horizontal and vertical displacements. These measurements are particularly intended to determine the small displacements of points representative of the behavior of the dam, its foundation, and abutments with respect to some stable frame. Geodetic surveys are often used for this purpose. The "absolute values" can be obtained only if the reference points are stable. Here, the GPS technique helps in establishing stations far enough from the dam to be outside of the deformation zone of the reservoir. In order to efficiently check their stability the number of reference points must be not less than 3,

preferably 4, for vertical control, and 4, preferably 6, for horizontal, and they should be connected together by observables, with as much redundancy as possible. Horizontal displacements in a critical direction, usually perpendicular to the axis of dam, can be surveyed with alignment techniques if the reference points of the alignment survey are stable or their movements can be determined by other techniques, for instance, by inverted plumb lines with a stable anchor point, or by geodetic methods. Vertical absolute displacements can be determined by geodetic leveling with respect to deeply anchored vertical borehole extensometers or to deep benchmarks located near the dam. Long leveling lines connecting the dam with benchmarks located several kilometers outside the deformation zone are not recommended due to the accumulation of errors.

(2) Relative movements. Deflections (inclinations) of a dam are usually measured by direct or inverted plumb lines. With reference to a horizontal line along the axis of the dam, different alignment methods are used in different levels of galleries to determine the relative movements between the blocks. Extensometers have now become important instruments for measuring differential foundation movements. A combination of geodetic leveling with suspended invar wires equipped with short reading scales at different levels of the dam and connected to borehole extensometers can supply all the needed information on the relative vertical movements as well as on the absolute vertical displacements and relative tilts.

(3) Foundation subsidence and tilts. They are measured with geodetic leveling, hydrostatic leveling, and tiltmeters. The last two are usually permanently installed in galleries.

(4) Strain measurements. Strain gauges are preferably embedded in the concrete during construction, installed on the faces of the dam after completion, or even embedded in foundation boreholes.

(5) Temperature. Temperature measurements should provide information on the thermal state of the concrete, water temperature at various levels, and atmospheric temperature. Temperature in the concrete is usually measured by telethermometers (thermistors, thermocouples, bi-metal thermometers) installed in the dam body.

(6) Uplift and leakage measurements. These measurements are generally carried out by nonspecific instruments. More elaborate devices may, however, be required to measure hydraulic pressure inside the rock. For

leakage measurements, it is important to combine an effective drainage system with these instruments.

(7) Joint measurements. Measurements are justified only in the case of joints separating two unsealed structures or to check grouting in dome or arch-gravity dams. Cracks are measured by the same methods, the instruments being installed on the surface.

(8) Water level measurements. Water level in a reservoir is one of the most important acting loads to a dam. For physical interpretation its measurement should coincide in time with the measurements of other deformation quantities.

c. *Identification of parameters to be observed in embankment dams.*

(1) Horizontal displacements. Horizontal displacements of the crest and other important points of embankment (berms, etc.) can be measured with geodetic methods and alignment. The comments made for concrete dams are also valid here. It is also possible to detect relative horizontal displacements of points inside the embankment by means of inclinometers.

(2) Groundwater and pore water pressures. Groundwater and pore pressures are very significant in monitoring earth dams. The pattern of seepage and pore water pressure, especially in the foundation and the impervious core, has a significant impact on the normal behavior of embankment dams. Since pore water pressures should not exceed design values, they must be carefully monitored, possibly with pressure cells. The greater the number of measurement profiles and the number of cells per profile, the more useful the data obtained will be.

(3) Settlements. For those occurring in accessible places, geodetic or hydrostatic leveling is customarily used to determine the settlements. The settlements of the foundation, or of interior structural parts which are not accessible (core, foundation contact), are detected through settlement gauges. The settlements of individual layers of the embankment should be monitored. This can be done through settlement gauges installed in the different layers.

(4) Total pressure measurements. It is sometimes necessary to check the total pressure inside the embankment or between the embankment and the foundation or adjacent structures.

(5) Water level measurements. Water level in the reservoir is the most important load on an earth dam,

causing horizontal movements and seepage. Its measurement should coincide in time with measurement of the deformations and seepage.

d. *Location of monitoring instruments.* In addition to the general guidelines given above, for gravity dams, each block should have at least one point. Tilts of the foundation should be measured at the center for small structures, and at not less than three points for larger structures.

(1) For multiple-arch and buttress dams, monitoring points should be located at the head and downstream toe of each buttress. In the case of massive buttresses and large arches, special attention should be paid to the foundations of the buttresses. If the buttresses are tranversed by construction joints, the behavior of joints should be observed.

(2) For arch-gravity dams and thick arch dams, absolute displacements of dam toe and abutments are critical. For small structures, the deformation of the central block is monitored. However, for large structures the measurement of deformations in each block is required.

(3) For thin arch dams, crest displacements in the horizontal and vertical are required. Special attention should be given to central cantilever, abutments, and abutment rock.

e. *Accuracy requirements.*

(1) No commonly accepted standards of accuracy requirements exist. As aforementioned, the accuracy at 95 percent probability should be equal to at least 0.25 of the maximum expected deformation regular behavior, and as high as possible for a discovered irregularity.

(2) For concrete dams, the accuracy for monitoring both horizontal and vertical displacements should typically range between 5 and 10 mm. For earth-rockfill dams, the accuracy should be about 20-30 mm for horizontal displacements and 10 mm for settlements during construction and operation. During operation, accuracies of 10-20 mm are normally required.

f. *Frequency of measurements.* The frequency of periodic measurements depends on the age of the structure and type of the monitoring system. If a fully automatic data acquisition system is used, the frequency of measurements does not impose any problem because the data can be decoded at any preprogrammed time intervals without any logistic difficulties and, practically, at no difference in

the cost of the monitoring process. However, in most cases, the fully automated systems are not yet commonly used and the frequency of measurements of individual observables must be carefully designed to compromise between the actual need and the cost. ICOLD gives the following general guidelines:

(1) Before and during construction, it may be useful to carry out some geodetic and piezometric measurements of the abutments.

(2) All measurements should be made before the first filling is started (initial operation). The dates of the successive measurements will depend on the level the water has reached in the reservoir. The closer the water is to the top level, the shorter will be the interval between the measurements. For instance, one survey should be conducted when the water reaches one-fourth of the total height; another survey when the water reaches midheight; one survey every tenth of the total height for the third quarter; one survey every 2 m of variation for the fourth quarter. Moreover, the interval between two successive surveys should never exceed a month until filling is completed.

(3) During the operation of the structure, measurements should be more frequent in the years immediately following the first filling when active deformation is in progress. For instance, geodetic surveys (more labor intensive) can be carried out four times a year, and other geotechnical measurements can be made once every 1 to 2 weeks.

(4) After the structure is stable, which takes usually 5 to 10 years or more, the above frequencies can be reduced by half. Not only the frequencies of measurement, but also the number of instruments read can be reduced according to what is learned during the first years of operation.

9-26. Optimal Design of the Configuration and Accuracy of the Monitoring Schemes

a. The general guidelines and restrictions regarding the locations and accuracy of instrumentation, discussed above, still give room for choice (within the general guidelines) of the final positions of, at least, some observation stations and accuracy of the observations. This concerns mainly the geodetic monitoring reference networks, which may provide different solutions for the accuracy of the observed displacements depending on the selected configuration of the connecting surveys, location of the reference stations, and type of observables (e.g.,

distances as opposed to angles, or an optimal combination of both).

b. The optimum design of geodetic positioning networks has been the subject of intensive investigations and publications by many authors over the past two decades. The optimization of geodetic positioning networks is aimed at obtaining the optimum positions of the geodetic points with the optimum accuracy, reliability, and economy of the survey scheme taken as the design criteria. Design of deformation monitoring schemes is more complex and differs in many respects from the design of positioning networks. The design is aimed at obtaining optimum accuracies for the deformation parameters rather than for the coordinates of the monitoring stations using various types (geodetic and nongeodetic) observables with allowable configuration defects. The sensitivity of the monitoring scheme to detect deformations is introduced as more general than the accuracy design criterion. Very recently, a separability concept has been added to the design criteria (separation between different possible deformation models).

c. There are practically two distinct optimal design methods:

- Analytical.

- Computer simulation ("trial and error").

Both methods usually involve an iteration process. The difference between them is that the former does not require human intervention and provides, mathematically, the optimum results, while the computer simulation method (CSM) provides acceptable results but they are not necessarily optimal. The CSM requires the experience of the designer but it can solve all the design problems while the analytical methods have been limited to some particular solutions only.

d. Recently, a CSM has been developed which does not require any human intervention during a fully automatic computational process from the moment of imputting the initial data through the iterative step-by-step upgrading of the design to the final design output. Although the underlying theory for the design of geodetic networks has been developed quite extensively, its full power of practical application has not been demonstrated in any real-life examples. An efficient algorithm has not existed until very recently. The major problem in this area was the inability of solving nonlinear matrix equations involved in the network design. This problem has recently been solved at the University of New Brunswick

(UNB) by developing a multiobjective analytical design methodology which allows for a fully analytical, multi-objective optimal design (optimal accuracy and sensitivity, optimal reliability, and optimal economy) of integrated deformation monitoring schemes with geodetic and geo-technical instrumentation. The method allows for a simultaneous solution for the optimal configuration and accuracy of the monitoring scheme according to the given criteria and restrictions concerning the locations of some observation stations and required accuracy of the defor-mation parameters.

9-27. Analysis of Deformation Surveys

Even the most precise monitoring surveys will not fully serve their purpose if they are not properly evaluated and utilized in a global integrated analysis as a cooperative interdisciplinary effort. The analysis of deformation sur-veys includes:

- Geometrical analysis which describes the geome-trical status of the deformable body, its change in shape and dimensions, as well as rigid body movements (translations and rotations) of the whole deformable body with respect to a stable reference frame or of a block of the body with respect to other blocks, and

- Physical interpretation which consists of: (a) a statistical (stochastic) method, which analyzes through a regression analysis the correlations between observed deformations and observed loads (external and internal causes producing the deformation), and (b) a deterministic method, which utilizes information on the loads, properties of the materials, and physical laws governing the stress-strain relationship which describes the state of internal stresses and the relationship between the causative effects (loads) and deformations.

a. Once the load-deformation relationship is estab-lished, the results of the physical interpretation may be used for the development of prediction models. Through a comparison of predicted deformation with the results of the geometrical analysis of the actual deformations, a better understanding of the mechanism of the deforma-tions is achieved. On the other hand, the prediction models supply information on the expected deformation, facilitating the design of the monitoring scheme as well as the selection of the deformation model in the geometrical analysis. Thus, the expression integrated analysis means a determination of the deformation by combining all types of measurements, geodetic and geotechnical, even if

scattered in time and space, in the simultaneous geometri-cal analysis of the deformation, comparing it with the prediction models, enhancing the prediction models which, in turn, may be used in enhancing the monitoring scheme. The process is iteratively repeated until the mechanism of deformation is well understood and any discrepancies between the prediction models and actual deformations are properly explained.

b. Recently, the concept of global integration has been developed in which all three — the geometrical analysis of deformation and both methods of the physical interpretation — are combined into a simultaneous solu-tion for all the parameters to be sought. The method still requires further elaboration, software development, and practical testing and, therefore, is not described in detail in this manual.

c. The deterministic and statistical modeling of deformations have been used in the analysis of dam deformations, at least in some countries, for many years. As aforementioned, the geometrical analysis has been done so far in a rather primitive way with geotechnical/ structural engineers analyzing separately the geotechnical observation data and surveyors taking care of the geodetic survey observations. The geotechnical analyses have usually resulted only in a graphical display of temporal trends for individual observables and the geodetic analysis would result in a plot of displacements obtained from repeated surveys which, very often, would not be even properly adjusted and analyzed for the stability of the reference points. Over the past ten years, an intensive study by the FIG working group has resulted in the devel-opment of proper methods for the analysis of geodetic surveys and has led to the development of the so-called UNB Generalized Method of the geometrical deformation analysis which can combine any type of observations (geotechnical and geodetic) into one simultaneous analy-sis. The developed methodology for the geometrical analysis is given below followed by brief descriptions of the statistical and deterministic methods used in modeling the load-deformation relationship. Finally, the concept of the hybrid physical analysis in which the statistical model-ing is combined with the deterministic method is described in more detail.

9-28. Geometrical Analysis of Deformation Surveys

a. Identification of unstable reference points. In most deformation studies, the information on absolute movements of object points with respect to some stable reference points is crucial. One problem which is

frequently encountered in practice in the reference networks is the instability of the reference points. This may be caused either by wrong monumentation of the survey markers or by the points being located still too close to the deformation zone (wrong assumption in the design about the stability of the surrounding area). Any unstable reference points must be identified first before the absolute displacements of the object points are calculated. Otherwise, the calculated displacements of the object points and subsequent analysis and interpretation of the deformation of the structure may be significantly distorted. Given a situation where points A, B, C, and D are reference points used to monitor a number of object points on a structure; if point B has moved but this is not recognized and it is used with point A to identify the common datum for two survey campaigns, then all the object points and reference points C and D will show significant changes in their coordinates even when, in reality, they are stable.

(1) Over the past two decades several methods for the analysis of reference networks have been developed in various research centers within the activity of the aforementioned FIG Study Group. There are basically two schools of thought. One is based on the congruency test, and the other is based on defining a datum for the second epoch of measurements which is robust to unstable reference points. In the first case, a failure in the congruency test is followed by a search for the new congruency test which has a minimum statistic. The test statistics are calculated by removing points, one by one in turn, from the set of reference points until all the unstable points are identified. In the second case, a method has been developed at UNB which is based on a special similarity transformation which minimizes the first norm of the vector of displacements of the reference points. The approach can be performed easily for one-dimensional reference networks and by an iterative weighting scheme for multi-dimensional reference networks until all the components of the displacement vectors satisfy the condition: $\sum | d_i |$ = minimum. In each solution, the weights are iteratively changed to be $p_i = 1/d_i$. After the last iteration, the displacement vectors that exceed their error ellipses at 95 percent probability identify the unstable reference points. The displacements obtained from the iterative weighted transformation are, practically, datum independent; i.e., that whatever minimum constraints have been used in the least squares adjustment of the survey campaigns, the display of the transformed displacements will always be the same. Thus the obtained graphical display represents the actual deformation trend which is used later on in selecting the best fitting deformation model, as described below.

(2) Several software packages for geometrical analysis have been developed, for example, DEFNAN, PANDA, and LOCAL. Some of them (e.g., DEFNAN) are applicable not only to the identification of unstable reference points, but to the integrated analysis of any type of deformations (to be discussed below), while others are limited to the analysis of reference geodetic networks only.

b. UNB Generalized Method for geometrical deformation analysis. In order to be able to utilize any type of geodetic and geotechnical observations in a simultaneous deformation analysis, the UNB Generalized Method of the geometrical analysis has been developed. The method is applicable to any type of geometrical analysis, both in space and in time, including the earlier discussed detection of unstable reference points and the determination of strain components and relative rigid body motion within a deformable body. It allows utilization of different types of surveying data (conventional surveys and GPS measurements) and geotechnical/structural measurements. It can be applied to any configuration of the monitoring scheme as long as approximate coordinates of all the observation points are known. In practical applications, the approach consists of three basic processes:

- Identification of deformation models.

- Estimation of deformation parameters.

- Diagnostic checking of the models, and final selection of the "best" model.

A brief description of the approach is given below.

(1) The change in shape and dimensions of a 3-D deformable body is fully described if 6 strain components (3 normal and 3 shearing strains) and 3 differential rotations at every point of the body are determined. These deformation parameters can be calculated from the well-known strain-displacement relations if a displacement function representing the deformation of the object is known. Since, in practice, deformation surveys involve only discrete points, the displacement function must be approximated through some selected deformation model which fits the observed changes in coordinates (displacements), or any other types of observables, in the statistically best way. The displacement function may be determined, for example, through a polynomial approximation of the displacement field.

(2) The displacement function can be expressed in matrix form in terms of a deformation model **Bc** as:

$$\mathbf{d}(x,y,z,t\text{-}t_o) \;=\; (u,v,w)^T = \mathbf{B}(x,y,z,t\text{-}t_o)\,\mathbf{c} \qquad (9\text{-}4)$$

where $\mathbf{d}$ is the displacement of a point (x,y,z) at time t in respect to a reference time t_o; u, v, and w are components of the displacement function in the x-, y-, and z- directions, respectively, $\mathbf{B}$ is the deformation matrix with its elements being some selected base functions, and $\mathbf{c}$ is the vector of unknown coefficients (deformation parameters).

(3) For illustration, examples of typical deformation models (displacement functions) in 2-D analysis are given below.

(a) Single point displacement or a rigid body displacement of a group of points, say, block B with respect to block A. The deformation model is expressed in the form of the following displacement functions:

$$u_A = 0,\; v_A = 0;\; u_B = a_o \text{ and } v_B = b_o \qquad (9\text{-}5)$$

where the subscripts represent all the points in the indicated blocks.

(b) Homogeneous strain in the whole body and differential rotation. The deformation model is linear and it may be expressed directly in terms of the strain components $(\varepsilon_x, \varepsilon_y, \varepsilon_{xy})$ and differential rotation, ϖ, as:

$$u = \varepsilon_x\, x + \varepsilon_{xy} y - \varpi y$$

$$v = \varepsilon_{xy}\, x + \varepsilon_y\, y + \varpi x \qquad (9\text{-}6)$$

(c) A deformable body with one discontinuity, say, between blocks A and B, and with different linear deformations in each block plus a rigid body displacement of B with respect to A. Then the deformation model is written as:

$$u_A = \varepsilon_{xA} x + \varepsilon_{xyA} y - \varpi_A\, y$$

$$v_A = \varepsilon_{xyA} x + \varepsilon_{yA} y + \varpi_A x \qquad (9\text{-}7)$$

and

$$u_B = a_0 + \varepsilon_{xB}(x - x_0) + \varepsilon_{xyB}(y - y_0) - \varpi_B(y - y_0)$$

$$v_B = b_0 + \varepsilon_{xyB}(x - x_0) + \varepsilon_{yB}(y - y_0) + \varpi_B(x - x_0) \qquad (9\text{-}8)$$

where x_0, y_0 are the coordinates of any point in block B.

(4) Usually, the actual deformation model is a combination of the above simple models or, if more complicated, it is expressed by non linear displacement functions which require fitting of higher order polynomials or other suitable functions. If time-dependent deformation parameters are sought, then the above deformation models will contain time variables.

(5) A vector Δl of changes in any type of observations, for instance, changes in tilts, in distances, or in observed strain, can always be expressed in terms of the displacement function. For example, the relationship between a displacement function and a change ds in the distance observed between two points i and j in two monitoring campaigns may be written as:

$$ds_{ij} = [(x_j - x_i)/s]\,u_j + [(y_j - y_i)/s]\,v_j -$$

$$[(x_j - x_i/s)]\,u_i - [(yj - yi)/s)]\,v_i \qquad (9\text{-}9)$$

where u_j, v_j, u_i, and v_i are components of the displacement function at points $(x_j,\ y_j,$ and $x_i, y_i)$, respectively. For a horizontal tiltmeter, the change $d\tau$ of tilt between two survey campaigns may be expressed in terms of the vertical component (w) of the displacement function as:

$$d\tau = (\partial w/\partial x)\ \sin\alpha + (\partial w/\partial y)\ \cos\alpha \qquad (9\text{-}10)$$

where α is the orientation angle of the tiltmeter.

(6) The functional relationships for any other types of observables and displacement functions are, in matrix form, written as:

$$\Delta l = \mathbf{AB}_{\Delta l}\,\mathbf{c} \qquad (9\text{-}11)$$

where $\mathbf{A}$ is the transformation matrix (design matrix) relating the observations to the displacements of points at which the observations are made, and $\mathbf{B}_{\Delta l}$ is constructed from the above matrix $\mathbf{B}(x,\ y,\ z,\ t\text{-}t_o\ )$ and related to the points included in the observables.

(7) If redundant observations are made, the elements of the vector $\mathbf{c}$ and their variances and covariances are determined through least-squares approximation, and their statistical significance can be calculated. One tries to find the simplest possible displacement function that would fit to the observations in the statistically best way.

(8) The search for the "best" deformation model (displacement function) is based on either a priori knowledge of the expected deformations (for instance from the finite element analysis) or a qualitative analysis of the deformation trend deduced from all the observations taken together. In the case of the observables being the relative displacements obtained from geodetic surveys, the

iterative weighted transformation of the displacements gives the best picture of the actual deformation trend helping in the spatial trend analysis. In the case of a long series of observations taken over a prolonged period of time, plotting of individual observables versus time helps to establish the deformation trend and the deformation model in the time domain. In the analysis, one has to separate the known deformation trend from the superimposed investigated deformation. For example, in order to distinguish between the cyclic (seasonal) thermal expansion of a structure with a one-year period of oscillation and a superimposed deformation caused by other effects which are, for instance, linear in time, all the measurements can be analyzed through a least-squares fitting of the cyclic function

$$y = a_1\cos(\omega t) + a_2\sin(\omega t) + a_3 t + a_4 + a_5\delta(t_i) + ..., \quad (9\text{-}12)$$

to the observation data, where $\omega = 2\pi/\text{yr}$, and a_3 is the rate of change of the observation (extension, tilt, inclination, etc.). The amplitude and phase of the sinusoid can be derived from a_1 and a_2. The constant a_4 is the y-intercept and the constants a_5, ... are possible slips (discontinuities) in the data series and $\delta(t_i)$ is the Kronecker's symbol which is equal to 1 when $t \geq t_i$, with t_i being the time of the occurrence of the slip, and is equal to 0 when $t < t_i$.

(9) Summarizing, the geometrical deformation analysis using the UNB Generalized Method is done in four steps:

(a) The trend analysis in space and time domains and the selection of a few alternative deformation models which seem to match the trend and that make physical sense.

(b) The least-squares fitting of the model or models into the observation data and statistical testing of the models.

(c) The selection of the "best" model that has as few coefficients as possible with as high a significance as possible (preferably all the coefficients should be significant at probabilities greater than 95 percent) and which gives as small a quadratic form of the residuals as possible.

(d) A graphical presentation of the displacement field and the derived strain field.

(10) The results of the geometrical analysis serve as an input into the physical interpretation and into the development of prediction models as discussed above.

9-29. Statistical Modeling of the Load-Displacement Relationship

a. The statistical method establishes an empirical model of the load-displacement relationship through the regression analysis, which determines the correlations between observed deformations and observed loads (external and internal causes producing the deformation). Using this model, the forecasted deformation can be obtained from the measured causative quantities. A good agreement between the forecasts and the measurements then tells us that the deformable body behaves as in the past. Otherwise, as in the previous case, reasons should be found and the model should be refined.

b. Interpretation by the statistical method always requires a suitable amount of observations, both of causative quantities and of response effects. Let d(t) be the observed deformation of an object point at time t. For a concrete dam, for example, it can usually be decomposed into three components:

$$d(t) = d_H(t) + d_T(t) + d_r(t) \quad (9\text{-}13)$$

where $d_H(t)$, $d_T(t)$, $d_r(t)$ are the hydrostatic pressure component, thermal component, and the irreversible component due to the nonelastic behavior of the dam, respectively. The component $d_H(t)$ is a function of water level in the reservoir and can be modeled by a simple polynomial:

$$d_H(t) = a_0 + a_1 H(t) + a_2 H(t)^2 + ... + a_m H(t)^m \quad (9\text{-}14)$$

where H(t) is the elevation of the water in the reservoir. The component $d_T(t)$ can be modeled in various ways depending on the information on hand. If some key temperatures $T_i(t)$, (i = 1, 2, ..., k) in the dam are measured, then

$$d_T(t) = b_1 T_1(t) + b_2 T_2(t) + ... + b_k T_k(t) \quad (9\text{-}15)$$

If air temperature is used, the response delay of concrete dams to the change in air temperature should be considered. If no temperature is measured, the thermal component can be modeled by a trigonometric function.

c. The irreversible component dr(t) may originate from a nonelastic phenomena like creep of concrete or creep of rock, etc. Its time-dependent behavior changes from object to object. It may be modeled, for example, with an exponential function. The following function is appropriate for concrete dams:

$$d_r(t) = c_1 t + c_2 \ln t \qquad (9\text{-}16)$$

The coefficients a_i, b_i, c_i in the above equations are determined using the least-squares regression analysis. The final model suggests the response behavior of the different causative factors and is used for prediction purposes.

d. For an earth dam, the thermal effect is immaterial and the irreversible component becomes dominant. It should be mentioned that the statistical method for physical interpretation is applicable not only to observed displacements, as discussed above, but also to other monitored quantities, such as stress, pore water pressure, tilt of the foundation, etc. The only difference is that the response function for each causative quantity may change.

9-30. Deterministic Modeling of the Load-Deformation Relationship

a. The deterministic method provides information on the expected deformation from the information on the acting forces (loads), properties of the materials, and physical laws governing the stress-strain relationship.

b. Deformation of an object will develop if an external force is applied to it. The external forces may be of two kinds: surface force, i.e., forces distributed over the surface of the body, and body forces, which are distributed over the volume of the body, such as gravitational forces and thermal stress. The relation between the acting forces and displacements d is discussed in many textbooks on mechanics. Let **d** be the displacement vector at a point and **f** be the acting force. They are related as

$$\mathbf{L^T D L d} + \mathbf{f} = 0 \qquad (9\text{-}17)$$

where **D** is the constitutive matrix of the material whose elements are functions of the material properties (e.g., Young's modulus and Poisson's ratio) and **L** is a differential operator transforming displacement to strain. If initial strain ε_0 and initial stress σ_0 exist, the above equation becomes

$$\mathbf{L^T D L d} + (\mathbf{L^T}\sigma_0 - \mathbf{L^T D}\varepsilon_0) + \mathbf{f} = 0 \qquad (9\text{-}18)$$

In principle, when the boundary conditions, either in the form of displacements or in the form of acting forces, are given and the body forces are prescribed, the differential equation can be solved. However, direct solution may be difficult, and numerical methods such as the finite element or boundary element of finite differences methods are used. The finite element method (FEM) is the most commonly used method in structural and geotechnical engineering, particularly in modeling dam deformations.

c. The basic concept of the FEM is that the continuum of the body is replaced by an assemblage of small elements which are connected together only at the nodal points of the elements. Within each element a displacement function (shape function) is postulated and the principle of minimum potential is applied, i.e., the difference between the work done by acting forces and the deformation energy is minimized. Therefore, the differential operator **L** is approximated by a linear algebraic operator. Numerous FEM software packages are available in the market ranging significantly in prices depending on their sophistication and adaptability to various types of material behavior. One very powerful software package is FEMMA (Finite Element Method for Multidisciplinary Applications) developed at the UNB for 2-D and 3-D finite element elastic, visco-elastic, and heat transfer analyses of deformations. FEMMA has found many practical applications in dam deformation analyses, in tectonic plate movements, in ground subsidence studies, and in tunneling deformations.

d. In the deterministic modeling of dam deformations, the dam and its foundation are subdivided into a finite element mesh. The thermal component dT and hydrostatic pressure component dH are calculated separately. Assuming some discrete water level in the reservoir, the corresponding displacements of the points of interest are computed. A displacement function with respect to water level is obtained by least-squares fitting of a polynomial to the FEM-computed discrete displacements. Then, the displacements at any water level can be computed from the displacement function. In computation of the thermal components, the temperature distribution inside the structure should first be solved. Again, FEM could be used, based on some measured temperatures (boundary conditions). Both the coefficient of thermal diffusivity and the coefficient of expansion of concrete are required. The thermal components for the points of interest are calculated using FEM with computed temperature at each nodal point. The total deformation is the sum of these two components plus possible action of some other

forces, e.g., swelling of concrete due to alkali aggregate reaction which can also be modeled with FEM.

e. FEM is, certainly, a powerful tool in the deterministic modeling of deformations. One has to remember, however, that the output from the FEM analysis is only as good as the quality of the input and as good as the experience of the operator who must have a good understanding of not only the computer operation but, particularly, good knowledge in the mechanics of the deformable bodies. One should not treat FEM as a magic (black box) tool.

9-31. Hybrid Method of Deformation Analysis

a. As one can see from the above sections, interpretation by statistical methods requires a large amount of observations, both of causative quantities and of response effects. Thus the method is not suitable at the early stage of dam operation when only short sets of observation data are available. In addition, some portions of the thermal and hydrostatic pressure effects may not be separated by the statistical modeling if the changes in temperature and in the elevation of water in the reservoir are strongly correlated. The deterministic method proves very advantageous in these aspects. The deterministic method is of an a priori (design) nature. It uses the information on geometric shape and material properties of the deformable body and acting loads to calculate deformations. However, due to many uncertainties in deterministic modeling such as an imperfect knowledge of the material properties, possibly wrong modeling of the behavior of the material (particularly when a nonelastic behavior takes place), and approximation in calculations, the computed displacements may depart significantly from the observed values $d(t)$. In this case, if, for example, a suspicion is that the discrepancy is produced by uncertainties in Young's modulus of elasticity, E, and the thermal coefficient of expansion, α, the deterministic model can be enhanced by combining it with the statistical method, in the form

$$d(t) + v(t) = xd_H(t) + yd_T(t) + c_1 t + c_2 \ln t \qquad (9\text{-}18)$$

where $v(t)$ is the residual, $d_H(t)$ and $d_T(t)$ are the hydrostatic and thermal components, respectively, calculated from the deterministic modeling, and the last two terms take care of the possible irreversible component. The functional model for the irreversible component may vary and can be changed by examining the residuals. The unknowns x, y, c_1, c_2 are estimated from the observations using the least-squares estimation. The coefficient x is a function of Young's modulus and y is a function of the thermal expansion coefficient of concrete:

$$x = E_0/E \qquad (9\text{-}19)$$

$$y = \alpha/\alpha_0 \qquad (9\text{-}20)$$

where E_0 and α_0 are the values used in the deterministic modeling.

b. There must be a calibration of the constants of the material properties using the discrepancies between the measured displacements of a point at different epochs and that calculated from FEM. One must be aware, however, that if the real discrepancy comes from other effects than the incorrect values of the constants (e.g., non-elastic behavior), the model may be significantly distorted.

c. Recently, as aforementioned, a concept of a global integration has been developed, where all three — the geometrical analysis of deformations and both methods of physical interpretation — are combined. Using this concept, deformation modeling and understanding of the deformation mechanism can be greatly enhanced.

9-32. Automated Data Management of Deformation Surveys

a. Advantages and limitations of automation. In the total effort of deformation monitoring, the quality of the analysis of the behavior of the object being monitored depends on the location, frequency, type, and reliability of the data gathered. The data concerned are any geotechnical observable as well as any conventional geodetic observable (angle, distance, or height difference). Apart from the location and type of instrumentation, the frequency and reliability of the data can be enhanced by employing an "automatic" system of data gathering or acquisition and processing (including the deformation analysis). A data management system encompasses everything that happens to the data from the instant at which it is sensed to the time of analysis. Under ordinary circumstances, the interval of time between sensing and analysis may extend over several days or more. Under critical conditions, this may have to be nearly instantaneous in order to provide a warning, if necessary. The volume of data may consist of only several items (in the simplest routine investigation) to many hundreds or thousands (in very complex, critical situations, particularly if vibration behavior is of interest). The rate of sampling may be annually, monthly, weekly, daily, hourly, or even more frequently. The amount of human involvement may range from total (a "manual" system) to virtually none (an "automatic" system). Neither extreme is practical. A

manual system is labor intensive and liable to errors or blunders and is less flexible in the re-examination of data. An automatic system is attractive but has some limitations. Although a "data acquisition system" strictly involves the gathering of data, the phrase has been used by many to mean the whole system of data management. The weighed advantages and limitations of an automatic data acquisition system are summarized in the following two lists.

Advantages of an automatic data acquisition system are:

(1) Personnel costs for reading instruments and analyzing data are reduced.

(2) More frequent readings are possible.

(3) Retrieval of data from remote or inaccessible locations is possible.

(4) Instantaneous transmission of data over long distances is possible.

(5) Increased reading sensitivity and accuracy can be achieved.

(6) Increased flexibility in selecting required data can be provided.

(7) Measurement of rapid fluctuations, pulsations, and vibrations is possible.

(8) Recording errors are fewer and immediately recognizable.

(9) Data can be stored electronically in a format suitable for direct computer analysis.

The limitations of an automatic system are:

(1) A knowledgeable observer is replaced by hardware, i.e., less frequent "intelligent" visual inspections.

(2) An excess of data could be generated, leading to a failure in timely response.

(3) The data may be blindly accepted, possibly leading to a wrong conclusion.

(4) There could be a high initial cost and, possibly, a high maintenance cost.

(5) Often requires site-specific or custom components that may be initially unproven.

(6) Complexity may require an initial stage of debugging.

(7) Specialized personnel may be required for regular field checks and maintenance.

(8) A manual method is required as an alternative (backup).

(9) A reliable and continuous source of power is required.

(10) The system may be susceptible to damage by weather or construction activity.

With an appropriate compromise between manual and automatic functions, a properly designed and working system can readily minimize the effects of the limitations mentioned above. Therefore, the advantages of an automatic (really "semiautomatic") system easily outweigh its disadvantages.

b. The ENEL system. In the ENEL system (Italian Ente Nazionale per l'Energia Elettrica), there are two major subsystems: "Off-line" which serves as a central storage of all data and "On-line" in which most of the activity takes place. It is the On-line portion that is of interest and will be described here. Two minicomputers are involved and are linked for the teletransmission of data. The remote virtually duplicates the functions of the local, except for the actual capture of data. The local, "ESSDI/L," provides data acquisition, validation, processing, storage, and transmission to other sites. Also, it issues a warning if the observed effect differs from the expected effect (as derived from deterministic modeling) by more than an established tolerance.

c. The USBR system. This system is for the Calamus (Embankment) Dam managed by the U.S. Bureau of Reclamation. The main aspect of this system is the means of communication by telephone line or satellite link from the dam to various locations in the country.

d. The USACE system. This is a system used by the St. Louis District, with respect to automated acquisition, processing, and plotting of data. An example of typical dialogue encountered in using the minicomputer-based system is given. Another system is under development,

initially to deal with totally automating piezometers in embankment dams, particularly with respect to local and district communication.

e. The UNB system. A data management system was devised by UNB for use on an IBM PC AT compatible on-site microcomputer. It was created to replace a manual system already in use for several years. In the field, a programmed data collector provides for direct connection to (and sometimes control of) instrumentation and for the keyboard entry for other equipment. The system can also accommodate manually recorded data or data directly acquired from instrumentation. The raw data are contained in observation files, archived for security, and are processed or "reduced" (using calibration, test values, etc.) into data files which are then used by various analysis and display software.

(1) In the field, there is a check file that is either accessed during data collecting or available in hard copy. The check file contains expected values predicted from stochastic (statistical) analyses of the data files and thus provides for a warning in the field. A warning is also given in the processing if the currently processed value differs from the most recent value in the data file, beyond a set tolerance.

(2) The major advantage of the UNB system is that any data or derived data, whether geotechnical or geodetic (so long as it has been repeated in a suitable time series), can be brought together in the integrated deformation analysis of a structure. In the process, a time series is analyzed for trend, with the separation of seasonal and long-term behavior. In the absence of actual temperature information, the trend ("y," the change in the value of an observed or derived quantity) against time ("t," in years) is described by the following equation

$$y = a_1 \sin\omega t + a_2 \cos\omega t + a_3 t + a_4 + a_5 + ...$$

in which

ω = 2π since a period of 1 year is assumed

a_3 = the "rate" or long-term trend

a_5 .. = possible values of slips accounting for discontinuities in the data series (a_4 is also a slip, but it is required so that the fitting is not unduly constrained)

and from which the amplitude and phase can be derived to provide a comparison of seasonal behavior among the various measurement points in the structure. Done rigorously using the method of least squares, the fitting provides a full statistical analysis of the trend with the detection of outlying or erroneous data. It is possible to derive a new series from two original series or to create a series from repeated geodetic campaigns (e.g., tilt derived from leveling). The system can also show several series of data simultaneously, without fitting, to provide a graphical comparison of the series.

(3) The treatment of the data can be described as follows: The geodetic data are treated traditionally in campaigns for adjustment ("C.A.") and spatial trend analysis ("S.T.A."). Once the observations have been repeated a sufficient number of times, they can be treated as a time series ("T.S.A" time series analysis and plot, e.g., tilt from leveling; "S.S.A." spatial series analysis and plot, subsidence). Geotechnical series are treated in a similar manner ("T.S.P." time series analysis and plot, "S.S.P." spatial series analysis and plot, e.g., borehole profile changes). The time series analyses ("T.S.A." and "T.S.P.") are fitted with the sinusoid in the above equation to separate seasonal effects from the long-term trend. All of the trend analyses are automated by command files which are set up to control fitting and automated plotting of several series in succession. All of the data can be used together in simultaneous integrated geometrical analyses following the UNB Generalized Method (e.g., "D.M."). If desired, several series can be plotted simultaneously, without fitting. Since both the observation files and the data files are ASCII text files, they are accessible also through any text editor for manual entry or editing and can be input to other applications.

f. Desirable characteristics of an automated system. Overall, the desirable characteristics of a data management system for deformation surveys (including both geotechnical and geodetic observables) include:

(1) Data integrity (offering checks in the field and later processing).

(2) Data security (automatic archiving and regular data file backup).

(3) Automation of acquisition, processing, and analysis.

(4) Compatibility and integration with other observables.

(5) Flexibility in access to the data for possible manual entry and editing.

(6) Data openness (useable by other software).

(7) Flexibility in the system to be easily modified to accommodate additional instrumentation or other forms of analysis.

(8) On-site immediate access to data or any of the forms of analysis.

(9) Near-real time results of trend or other analyses.

(10) Testing and calibration is an integral component of the system.

9-33. Standards and Specifications for Dam Monitoring

a. Although there are significant differences between various countries in the quality of monitoring deformation, there is no one country which can serve as an example for others concerning all the three main aspects of dam deformation monitoring, i.e. monitoring techniques, design of monitoring schemes, and analysis and management of the collected observations.

b. A few countries, mainly eastern European countries, during the time when they were still part of the "communist block," including China, developed some national standards and specifications for dam deformations. Unfortunately, these specifications were developed for practically unrealistic conditions under government dictatorship and ownership of all dams in their countries. Although some of the standards and specifications may be technically acceptable, they would have to be carefully reviewed and extensively modified for practical use. Although there are some reputable books on specialized geodetic instrumentation, there is no up-to-date manual or book which discusses all of the aspects of geodetic monitoring surveys, particularly the design and processing of the geodetic monitoring networks.

c. Although it seems to be unrealistic and practically impossible to prepare overall detailed standards and specifications for dam monitoring at the national level, certain processes could, and perhaps should, be standardized, particularly:

(1) Calibration of instruments.

(2) Procedures for the integrated geometrical analysis and data management from the moment of the data collection, through the reduction of data and trend analysis (including the identification of the unstable reference points) to the determination of the deformation model.

d. General guidelines are required with respect to: design of the integrated monitoring schemes, particularly optimal choice of instrumentation, selection of the parameters to be monitored, the required accuracy, optimal location of the instruments, and frequency of measurements.

9-34. Monitoring Techniques and Their Applications

a. With the recent technological developments in both geodetic and geotechnical instrumentation, at a cost one may achieve almost any, practically needed, instrumental resolution and precision, full automation, and virtually real-time data processing. Remotely controlled telemetric data acquisition systems, working continuously for several months without recharging the batteries in temperatures down to -40°C are available at a reasonable cost. Thus, the array of different types of instruments available for deformation studies has significantly broadened within the last few years. This creates a new challenge for the designers of the monitoring surveys: what instruments to choose, where to locate them, and how to combine them into one integrated monitoring scheme in which the geodetic and geotechnical/structural measurements would optimally complement each other.

b. As far as the actual accuracy of deformation surveys is concerned, the main limiting factors are not the instrument precision but the environmental influences and negligence by the users, namely:

- Influence of atmospheric refraction (all optical and electro-optical measuring systems).

- Thermal influences, affecting the mechanical, electronic, and optical components of the instruments (in any type of instrumentation) as well as the stability of survey stations.

- Local instability of the observation stations (improper monumentation of survey stations and improper installation of the in situ instrumentation).

- Lack of or improper calibration of the instruments.

- Lack of understanding by the users of the sources of errors and of the proper reduction of the collected observations.

Most of the above-listed effects could be eliminated or drastically reduced if the monitoring surveys were in the hands of qualified professionals.

c. The problem of calibration is very often underestimated in practice, not only by the users but also by the manufacturers. For long-term measurements, the instrument repeatability (precision) may be affected by aging of the electronic and mechanical components resulting in a drift of the instrument readout. Of particular concern are geotechnical instruments for which the users, in general, do not have sufficient facilities and adequate knowledge for their calibration. The permanently installed instruments are very often left in situ for several years without checking the quality of their performance.

d. The last aspect, the lack of understanding of the sources of errors affecting various types of measurements and the proper data handling is, perhaps, the most dangerous and, unfortunately, the most frequent case in measurements of deformations. The measurements and processing of the monitoring data, particularly geodetic surveys, are usually in hands of technicians who may be experienced in the data collection, but have no educational background in handling and reducing the influence of various sources of errors. In this case, even the most technologically advanced instrumentation system will not supply the expected information.

e. A worldwide review of the monitoring techniques used in monitoring surveys indicates that, generally, there are no significant differences between different countries in the employed instrumentation. There are, however, differences in the accuracy requirements, the required frequency of observations, and in the details concerning the use of the instruments. This is also true among USACE districts. Strong biases toward either geodetic or geotechnical instrumentation have been developed in individual countries. The biases are correlated with the level of educational background in geodetic surveys. For instance, in Switzerland and Germany, where the standard of geodetic surveying education and the number of specialists in surveying engineering are high, the geodetic surveying techniques play the dominant role in monitoring large dams. Whereas, in the countries like Italy, France, and U.K., where the education in surveying engineering does not have a long-standing tradition, the geotechnical techniques are mostly used. ICOLD seems to be biased toward use of geotechnical/structural instrumentation rather than geodetic. This is dictated, perhaps, by the fact that the members of the ICOLD Committee on Monitoring Dams and Their Foundations are predominantly specialists in geotechnical instrumentation with some obsolete views on the use geodetic techniques. With the exception of a few individual dams, there is no country which takes full advantage of the optimal combination of both techniques, i.e., the concept of the integrated monitoring surveys developed within the activity of FIG. The biases toward one or another type of techniques are obviously produced by a lack of specialists with full knowledge of both geotechnical and geodetic measurements. This leads to the following:

(1) The concept of the integrated monitoring systems, in which the geotechnical and geodetic surveying techniques complement each other, should be made known to all the owners of large dams through the aforementioned efforts of the national committees on large dams and publication of guidelines.

(2) Monitoring schemes for all new dams should be designed at the design stage of the dam.

(3) As far as the new monitoring technologies are concerned, more research is still needed in:

(a) Optimal use of GPS.

(b) Automation of data acquisition including the optimal selection of electronic and optical transducers, comparison of their performance (sources of errors, durability), development of calibration methods, and

(c) Application of new technologies, for example the use of the optic-fibre sensors, CCD sensors, etc.

9-35. Analysis and Modeling of Deformations and Their Applications

a. Over the past 10 years there has been a significant progress in the development of new methods for the geometrical and physical analyses of deformation surveys. FIG has been leading in the developments, particularly in the areas of integrated geometrical analysis of structural deformations and combined integrated analysis. However, due to the aforementioned lack of the interdisciplinary cooperation and insufficient exchange of information, FIG developments have not yet been widely adapted in practice. Therefore, the general worldwide use of the geometrical analysis methods is still poor, including even the basic analysis of geodetic monitoring networks. The latter is of a particular concern in the United States where there

is a shortage of qualified surveying engineers. The worldwide situation with the physical analysis is much better with most countries who utilize both deterministic and statistical methods for modeling and interpreting deformations at various levels of sophistication. However, most countries do not take full advantage of the developments in the integration of the observed deformations with deterministic models to enhance the latter ones. Also only few countries utilize the observed deformations to develop prediction models through the regression analysis. Italy seems to lead in the use of the combined statistical and deterministic modeling. Canada, within the activities of FIG, leads in the development of new concepts in the global integrated analysis. For example, the UNB has developed new methodology and software for the integrated geometrical analysis, and for the finite element deterministic modeling and prediction of deformations.

b. The above comments lead to the following:

(1) The analysis of deformation surveys should be in hands of interdisciplinary teams consisting of geotechnical, structural, and surveying engineers specialized in both geometrical and physical analyses.

(2) More use should be made of the concepts and developed methodologies for the geometrical integrated analysis and combined deterministic/statistical modeling of deformations.